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The Capacity Requirements and Design of Distribution Reservoirs

By R. C. Kennedy

THE topography of the area lying adjacent to the east shore of San Francisco Bay presents problems in water distribution found in few metropolitan districts. The developed area slopes upward from sea level to elevation 1350 and then down again to about elevation 350, with many intermediate irregularities, and with numerous interruptions at right angles to the general slope by the valleys of streams flowing into the bay. One of the major fault lines of the state, traversing the area parallel to the shore line, is responsible for further complicating the topography. The result insofar as the water system is concerned, is that 67 distinct distribution zones, either direct or through pressure regulators, are necessary to maintain pressures within proper limits. In general, the terrain below elevation 250, which includes all of the industrial and commercial areas and considerable residential area, is served by gravity from four terminal storage reservoirs and by the aqueduct bringing water from the Pardee Reservoir, some 100 miles to the east in the Sierra Nevada foothills. To boost water to the higher residential zones, 41 pumping plants, each with a reservoir, are operated.

During the 12 years of operation of the district, an unusually large amount of work has been done and is still being done toward

A paper presented on October 25, 1940, at the California Section Meeting, Los Angeles, by R. C. Kennedy, Assistant Chief Engineer and Assistant General Manager, East Bay Municipal Utility District, Oakland, Calif.

the construction, enlargement and improvement of its distribution reservoirs. This new work has been necessary mainly due to the rapid increase in the number of water services in the East Bay area, and to the annexation of large suburban areas lying adjacent to the original nine cities served. In 1929, there were 117,600 active services of all types connected to the district's water mains. At the present time, this number is 145.200, an increase of 24 per cent in approximately eleven years. The total area in which service is offered is 179 sq. mi., 28 per cent more than at the time the district purchased the private company in December, 1928.

The greatly enlarged demands on the distribution system have been met by the installation of 770 mi. of new mains of which 465 mi. were replacements of existing inadequate lines and the remainder, extensions into areas not previously served. In addition, 51 wood or steel tanks and 10 pre-stressed concrete reservoirs were constructed or are now nearing completion. Several of the existing large earth reservoirs were improved with new concrete linings, outlet facilities, roofs, and other features. The distribution reservoir capacity has been increased by over 25 million gallons during this period to make a total capacity of 345 million gallons.

This paper is a description of the methods adopted by the district in determining storage capacities required in its distribution reservoirs and the features of their design, insofar as these vary from the usual standards.

"Depth-Over-Area" Method of Estimating Demand

To arrive at a uniform program of storage reservoir development for each zone, it was necessary first to determine a method for estimating ultimate requirements. The size of the site for the reservoir and the size of supply lines are based on these estimates. For already well-developed areas, reservoirs are made to ultimate capacity and of permanent type; but the storage for new subdivisions generally consists of temporary structures of moderate size, pending proof of need for greater capacity.

Studies of water use in areas already sufficiently built-up to serve as an indication of the consumption trend resulted in the adoption of the method suggested by C. H. Lee (Jour. A.W.W.A. **17:** 193 (1927)) for estimating consumption demands. In this method, which may be called the "depth-over-area" method, the consumption per year is expressed in terms of feet depth over the area in question,

as is common in irrigation practice, instead of in terms of gallons per capita per day. The depth of water consumed obviously will vary with many factors such as climate, class of development, size of building lots, type of soil, cost of water, and many other considerations; but for any given metropolitan district, the factors governing consumption are so reduced in number that residential and commercial zones may be divided into a small number of classifications, and the water consumption will not vary greatly in areas of the same class.

Application of Method

In arriving at the water depths to be employed in the consumption forecasts, the total quantity for one year, as recorded on the water meters, was used for each of the 339 meter-reading zones into which the district has been divided; and from these figures, the equivalent depths over the areas were computed. After it was apparent, from the study, that the basic method was sound, typical groups of meter reading zones, involving a total of 3,500 services, were selected and closer study was devoted to them. The consumption of each typical group was obtained for two years and corrections were made for the unoccupied lots, vacant houses, and areas of land rendered unusable by ravines or other natural features. The result represents the average yearly consumption for this class of development with the entire area fully developed. The amount was then increased by 10 per cent to allow for main leakage, evaporation from reservoirs and other unmeasured losses. The ratings, expressed in feet depth over the area per year, that were adopted after this study, and a brief description of each class of development, are as follows:

Class A, 2.0 feet. Large homes on lots of one-half acre or larger, with highly developed lawns and shrubbery, some swimming pools.

Class B, 1.5 feet. Moderate sized homes with well-developed lawns and shrubbery.

Class C, 1.1 feet. Low cost homes, on small lots, many of multiple dwelling type, with few or small yards and gardens.

It is found that not only does the annual consumption in zones of different classes vary materially, but that there also exists considerable variation between these zones in the monthly and daily consumption above or below yearly average. This is to be expected, since use for irrigation involves a greater increase in sections where lawns and gardens are well developed than where they are but minor items.

In Fig. 1 are plotted the percentages of consumption above or below the mean for each month of the year and for each of the three classes of zones under study. The Class A zone rises to a monthly peak of 59 per cent above the average. Similarly, the Class B area reaches 37 per cent, and Class C, 25 per cent over the monthly average.

For the maximum day, studies of pumping records indicate increases in the rate of consumption over the maximum month for the various zones to be about 40, 30 and 20 per cent, respectively. Nonconformity of pressure zones with the areas selected for classification

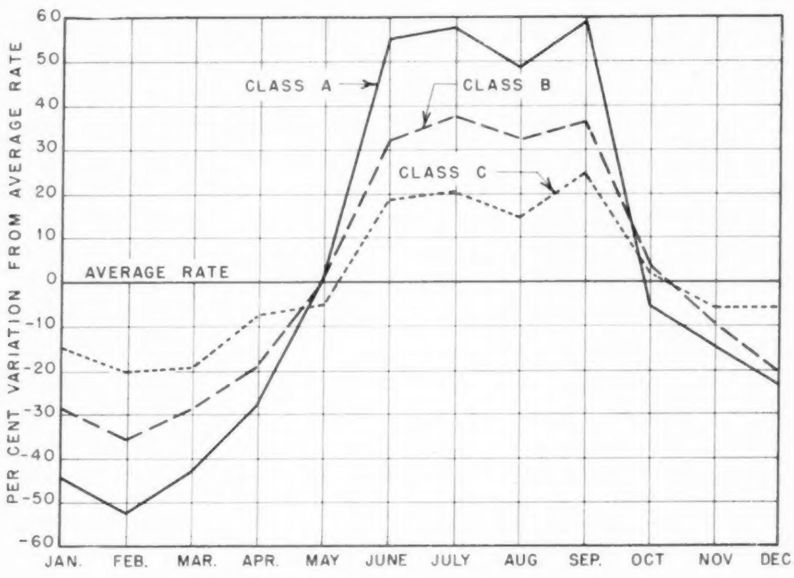

Fig. 1. Monthly Variation in Consumption

made the latter figures impossible of accurate determination. For the district as a whole, the increase is found to be 31 per cent, thus indicating that the approximations are not far from correct. Table 1 is a summary of these figures.

Studies of classes of development other than those above discussed showed great variations. Commercial districts are found to consume from 1.0 to 6.0 ft. depth per year, depending upon class and height of buildings. Apartment-house districts consume from 1.5 to 3.0 ft. depth, but this does not include areas built up solidly to large apartment houses such as are found in some cities. The method

used does not appear to be applicable to industrial districts owing to the non-uniformity of water demands in the various processes and to the use of wells. An oil refinery is the largest consumer of the entire district, but a ship building plant of comparable size in area uses water for sanitary purposes only.

In cities where distribution storage is relatively cheap due to natural elevations for reservoir sites, the peak draft at various hours of the day will not enter into the determination of the reservoir capacity required, since there will usually be provision for more than one day's supply. Such hourly peaks also are unlikely to be the determining factor in the sizes of mains other than transmission mains, as fire demands at any point in the distribution system will generally be higher than any possible normal consumption uses. In localities where storage must be elevated above the ground, due to

TABLE 1

Summary of Variations in Consumption by Consumer Classes

CLASS	ANNUAL CONSUMPTION DEPTH	MAX. MONTHLY INCREASE OVER ANNUAL AVERAGE	MAX. DAILY INCREASE OVER MAX. MONTHLY RATE	MAX. DAILY INCREASE OVER ANNUAL AVERAGE	MAX. DAILY CONSUMPTION OVER ANNUAL AVERAGE, AS USED IN DESIGN	DEPTH PER YEAR IF TAKEN AT RATE OF MAX. DAY
	ft.	%	%	%	%	*ft.*
A	2.0	59	40	123	125	4.5
B	1.5	37	30	78	80	2.7
C	1.1	25	20	50	50	1.7

flat topography, however, economy dictates that greater dependence be placed on continuous supply from the source, and that the reservoir be sufficient to balance demands for short periods only, such as a single day or less. For these conditions, data on variations of flow during the day are of interest.

For the purpose of determining the flow characteristics during the day for a large zone of Class B residential development, service was delivered only from storage for three consecutive days (Thursday, Friday and Saturday) during a high consumption period. Gage readings each hour gave the data required to compute the rate of flow. It was found that the highest peak flows occur generally at from 6 to 7 P.M. in residential zones, and that a lesser peak occurs during the period of from 9 to 10 A.M. The afternoon increase rose to from 110 to 135 per cent above the average for the day, while the

morning peak was in no case over 80 per cent above the daily average. Figure 2 shows graphically the average hourly variation in rate of use in this zone.

Having adopted the basic rates as outlined herein, it is a simple matter to compute the annual demand for any zone, as soon as its area has been obtained and the class of development assumed. Obviously, the latter item requires considerable judgment, especially

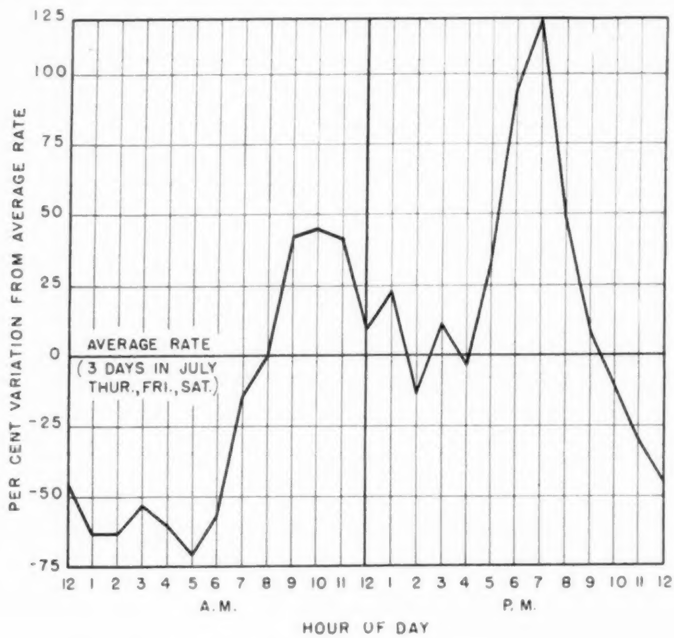

Fig. 2. Hourly Variation in Consumption

in new subdivisions. Building restrictions, accessibility, desirability of view and many other features will determine the ultimate classification.

The average daily use in gallons is given by the equation

$$Q = \frac{DA \times 43560 \times 7.5}{365} = 895\,DA,$$

in which D is the depth of use per year in feet and A is the net area in acres after deducting areas not usable for water consuming purposes, other than streets. The maximum daily use is taken as 125,

80 or 50 per cent above the average for the year, depending upon the classification assumed for the area.

Storage Requirements

Adequate storage for each distribution zone is generally of importance for one or more of several reasons, among which may be: (1) to create a reserve for short periods of emergency fire demands or curtailment of supply, thus giving flexibility to the distribution system; (2) due to the resulting two-way feed, to make possible the use of smaller mains and still maintain a proper limit on pressure variations during periods of high consumption; (3) to make possible minimum capacity for treatment plants or pumping facilities due to continuous operation; (4) to take advantage of lower electric power rates by pumping at off-peak hours; and (5) to decrease dead-end troubles.

In determining the volume of storage required for each zone, two items, consumption demand and fire demand, must be considered. In large zones the fire demand will be a minor consideration, but in small zones it may be the determining factor.

It is believed that the National Board of Fire Underwriters may safely be assumed to be the highest authority in the matter of water requirements for fire fighting. The board, in its present grading schedule, states that, as affecting reliability of supply, "... it appears to be a reasonable assumption that a *storage sufficient to provide fire flow for ten hours during a period of five days of maximum consumption* is sufficient to permit the making of most of the repairs, alterations or additions incident to the operation of a water supply system."

The fire flow that should be available is assumed by the board to vary with the population of the city as shown in Table 2.

In *residential* districts: "The required fire flow depends upon the character and congestion of the buildings. Sections where buildings are small and of low height and with about one-third the lots in a block built upon require not less than 500 g.p.m.; with larger or higher buildings up to 1,000 g.p.m. is required, and where the district is closely built, or buildings approach the dimension of hotels or high value residences, 1,500 to 3,000 g.p.m. is required, with up to 6,000 g.p.m. in densely built sections of three-story buildings."

The Underwriters are making a revision in their rules that will probably be issued in the early part of 1941. The new rule will

provide, under the items of adequacy and reliability, for a fire flow duration of four hours where the fire flow requirement is 1,000 g.p.m. or less, with an increase of one hour in the duration for each 250 g.p.m. increase in fire flow up to 2,500 g.p.m., corresponding to a population of 6,000, where ten hours fire flow is specified.

In using the depth-over-area method heretofore described, it has not been necessary to estimate the population density for other purposes, but some basis for estimating roughly the ultimate population is needed for use in determining the fire demand and hours duration of the same from the above rule. Studies of the residential areas of the East Bay cities indicate that the population density in the classifications described is approximately as follows: Class A, 12; Class B, 18; and Class C, 22 persons per acre. By use of these figures the

TABLE 2

Required Fire Flows for Average Cities by Population

POPULATION	REQUIRED FLOW	POPULATION	REQUIRED FLOW
	g.p.m.		*g.p.m.*
1,000	1,000	28,000	5,000
2,000	1,500	40,000	6,000
4,000	2,000	60,000	7,000
6,000	2,500	80,000	8,000
10,000	3,000	100,000	9,000
13,000	3,500	125,000	10,000
17,000	4,000	150,000	11,000
22,000	4,500	*200,000	12,000

* Over 200,000 population, 12,000 g.p.m., with 2,000 to 8,000 g.p.m. additional for a second fire.

population of the zone under consideration is readily estimated and hence the fire demands can be taken from the table.

Under the rates of flow specified, the Underwriters require that a minimum pressure of 20 pounds be maintained at the hydrant, to overcome friction loss in the hydrant branch, in the hydrant itself and in the suction hose to the pumper. In districts where hydrants conform to required spacing and are of satisfactory size and type, pressures of 10 pounds are permissible. Credits are given in these cases for pressures in excess of 20 or 10 pounds.

The requirements of the National Board of Fire Underwriters, as above set forth, are regarded as sufficiently high so that if they are met satisfactorily, no further concern need be felt for the requirements for consumption purposes. In cases where the rules are not

followed, however, the minimum condition is obviously that where the storage, augmented by a steady supply from the source, is sufficient to carry through a period beginning at the date when consumption becomes greater than the supply, and ending when equality is again reached. Some safety factor above this storage appears highly desirable in all cases, to allow for power shutdowns, main breaks, etc. The amount of extra storage to be constructed will be influenced by its cost, since there are usually other means, such as standby pumps or additional feeder mains, that can be substituted for storage.

Back-Feed Method of Adding to Distribution System Capacity

In areas where a series of pumping lifts is necessary, it is found to be good economy to place excess storage at or near the top, thus making possible back-feed to any lower zone during exceptionally high demands. In computing fire requirements for all lower zones, credit may then be taken for this excess storage, although in no case can a rate in excess of the actual capacity of the emergency connections from the higher zone be considered.

By this means, the fire demands, which otherwise would be the determining factor in small zones, can be taken care of with no provisions for extra capacity over that needed for consumption. As an example, a zone of Class B residential development, closely built, has an area of 300 acres. At the rate of 18 per acre, the population will be 5,400. From the proposed schedule, the fire demands will be about 2,250 g.p.m. for nine hours, or 1,215,000 gallons. The maximum daily consumption for this zone will be, from the equation previously stated, $Q = 1.5 \times 895 \times 300 \times 1.8 =$ 725,000 gallons. Pumps and mains are of such capacity that only 600,000 g.p.d. can be delivered into the zone from the source of supply. If the reservoir is full at the beginning of the five-day period of maximum consumption, it will be drawn down by consumption requirements to the extent of 625,000 gallons at the end of this period. During the last nine hours, the additional assumed use for fire fighting will further deplete the storage by 1,215,000 gallons. Thus, the minimum storage for this zone, to comply with the Underwriters' requirements, will be 1,840,000 gallons. If, however, storage to take care of the fire demands is available from a higher zone, and if mains are of adequate capacity, the reservoir need have a capacity of only 625,000 gallons, a reduction of about two-thirds.

The latter arrangement may require somewhat larger mains, to carry the back-feed, than otherwise would be necessary. In case of very long pump lines, this fact may make it more economical to provide sufficient storage for fire and consumption demands in each zone.

Design of Reservoirs

Within the distribution system of the East Bay Municipal Utility District are examples of most of the usual types of reservoirs. Greatest in number and capacity, however, are concrete-lined earth, pre-stressed concrete and welded-steel reservoirs. Numerous small temporary redwood tanks have been constructed in newly developed sections.

A great deal of effort and expense have been devoted to making these structures architecturally suited to the area in which they are located and to the landscaping and maintenance of grounds. While in industrial sections, steel tanks or reservoirs are acceptable, in residential zones, all new permanent storage is in either concrete-lined earth reservoirs or pre-stressed concrete reservoirs.

Earth Reservoirs

One of the principal problems in connection with an earth reservoir is the roof. While it is recognized that such structures should be covered, there is bound to be great opposition among nearby residents to roofing a reservoir that has previously been open, unless the roof be covered with earth deeply enough to support lawns and shrubs. This involves a heavy concrete supporting structure whose cost is greatly in excess of the cost of a light wooden roof, even after due allowance is made for the shorter life of the wood. The result of these considerations is that the five largest earth reservoirs remain open, and while they are a great credit scenically to the neighborhoods in which they are located, they require regular copper-sulfate treatment, in the summer months, to keep them free from algae.

As a means of practical protection and as a psychological factor, open reservoirs recently remodeled are equipped with both a 6-foot chain link fence on the property line and a 4-foot fence of similar type along the parapet wall, which is kept well back into the property where possible. These reservoirs are lined with 4-inch slabs, with continuous reinforcement of $\frac{3}{8}$-inch round bars at 12-inch spacing, both ways over the bottom and over sloping sides. Although fine

contraction cracks are not uncommon in these slabs, the total leakage is negligible.

In the 37-million gallon Summit Reservoir (Fig. 3), on which improvements were recently completed, a dividing wall 6 ft. high was installed to make cleaning possible without putting the entire structure out of service. By placing a wall of the same height, but at right angles to the main wall, within the concrete operating tower, a simple arrangement of gates was made to produce the necessary flexibility of operation. To provide freeboard, and for architectural

FIG. 3. Summit Reservoir; capacity, 37 mil. gal.

effect, a stone-masonry parapet wall, 2 ft. high, was placed around the reservoir at the top of the concrete lining.

Pre-Stressed Concrete Reservoirs

Most of the new storage structures built in residential sections have been of pre-stressed concrete (Fig. 4). The district has built ten of these structures, ranging from 200,000 to 3,500,000 gallons capacity. Certain improvements have been made in the conventional design—largely in the reinforcing bands and in the joint between wall and floor.

The reservoirs are constructed with a continuously-reinforced

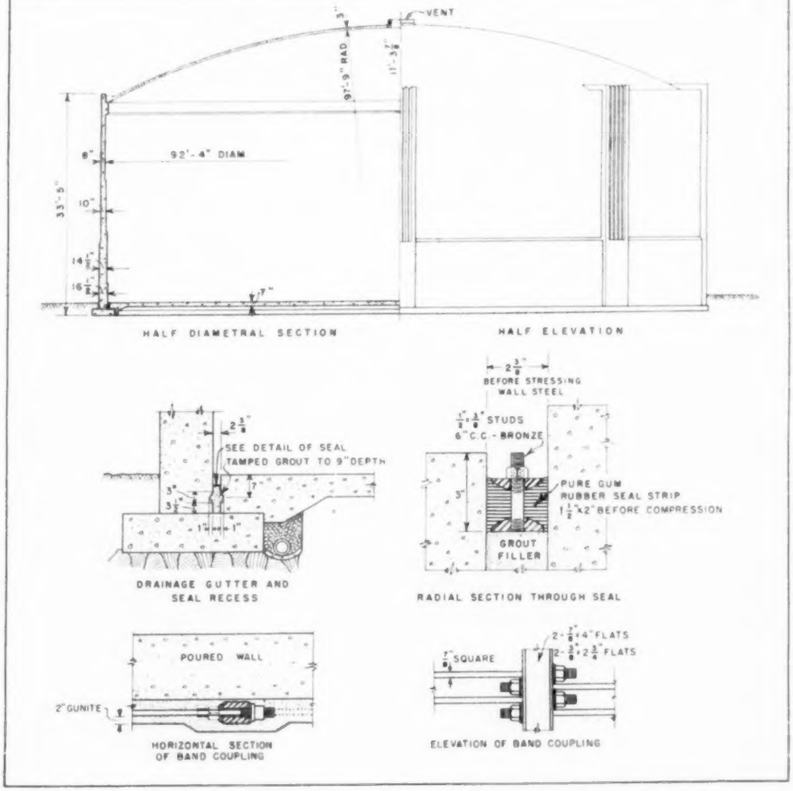

FIG. 4. Detail of Reinforcement on Pre-Stressed Concrete Reservoir

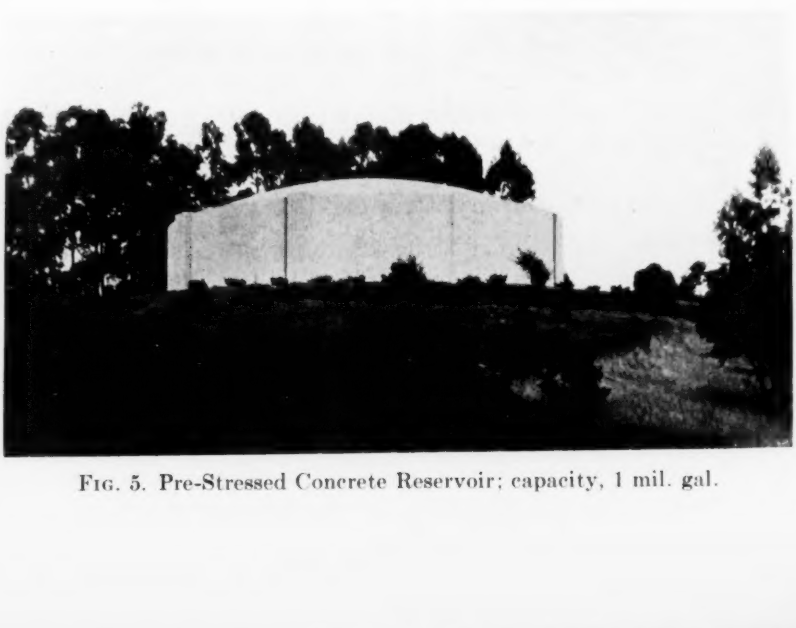

FIG. 5. Pre-Stressed Concrete Reservoir; capacity, 1 mil. gal.

concrete bottom slab, and with side walls poured in units, each entirely free from its neighbors, and extending from bottom to top. The edges of the units are painted with a 1:3 mixture of iron dust and cement before the next pour is made.

When all of the wall units are in place and the forms removed, the reinforcement is applied. This consists of $\frac{7}{8}$-inch square rods with $1\frac{1}{4}$-inch upset and threaded ends, that connect vertical steel beams spaced at intervals of approximately 40 ft. around the structure. The beams are of box type, built up by welding, and drilled with holes of proper spacing for the rods. The rods are gradually tightened by nuts until all are at the desired stress. The result is a steel "corset" that places the concrete wall under sufficient compression so that when full of water, a predetermined residual compression remains. A detail of this reinforcement is shown in Fig. 5. The principal advantage of the corset type reinforcement is that three men can handle the bars, whereas with turnbuckled bands, 15 to 20 men were required to place them on the walls. The square steel bands give improved bearing against the concrete, and prevent rolling as the bands are tightened.

By use of special manganese steel for the bands, it is found that considerable saving results over the cost of the formerly used mild steel. The specifications require an elastic limit of 55,000 and an ultimate strength of 85,000 lb. per sq.in., with an elongation in 8 in. of 17 per cent minimum. The bands are stressed so that when the structure is full of water they will reach 30,000 lb. per sq.in. The maximum concrete stress when the reservoir is empty is 500 lb. per sq.in. Experiments with still higher strength steels for the bands have indicated that there is no particular difficulty in obtaining the metal, but that it does not thread smoothly.

Stresses in the bands are accurately fixed by means of a wrench, specially built for this purpose, which incorporates a jointed handle so designed that a 6D copper nail is sheared when proper torque is reached. When all bands are at proper stress, wire-mesh reinforcement is applied and the surface is coated with gunite to a thickness of $\frac{3}{4}$ in. over the steel. Where necessary, a second layer of bands and gunite is applied over the first.

The roofs of the reservoirs up to 93 ft. in diameter are of the dome type, 3 in. thick. Larger reservoirs have flat slab roofs with square supporting columns. The largest of such structures built by the district to date is 160 ft. in diameter, 30 ft. high, and contains $3\frac{1}{2}$ million gallons.

The annular space between the edge of the floor and the side wall, $2\frac{3}{8}$ in. wide and 12 in. deep, is left open until the walls are complete. It is then filled with a rich cement mortar to within 4 in. of the top. The mortar fills key grooves that tie the floor and wall against relative movement. The water seal which occupies the remainder of the annular space, consists of a pure rubber strip of rectangular section, $1\frac{1}{2}$ by 2 in., compressed between two half-oval steel strips by means of $\frac{1}{2}$-inch bronze studs, at 6-inch centers.

Reservoirs constructed according to this design are found to be bottle-tight.

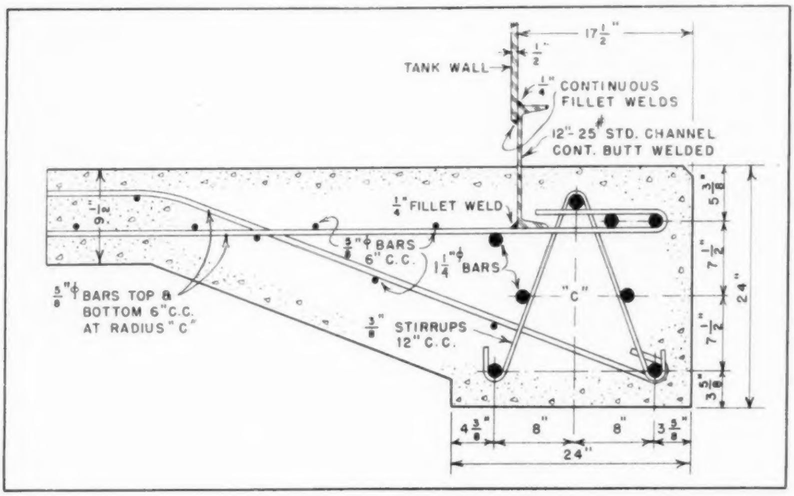

Fig. 6. Detail of Concrete Bottom of Steel Reservoir; showing imbedded structural steel channel

Steel Reservoirs

The steel reservoirs constructed by the district differ from the standard welded American Petroleum Institute design only in the fact that concrete instead of steel bottoms are used. The object of this change is to avoid difficulty from corrosion of the bottom membrane, particularly on the under side where it cannot be repainted, and to secure proper anchorage of the side wall to the foundation. The latter is of particular importance in areas where provision against earthquakes must be made.

The method adopted for eliminating the bottom membrane is to imbed a structural steel channel, bent to the curve of the side plates, into the concrete, as shown in Fig. 6, and to weld the side plates to this channel. The radial reinforcing bars of the base are also welded to the channel, and the concrete is further reinforced by circumferential bars and stirrups.

Thirteen of these tanks or reservoirs have been built by the district during the past eight years, in sizes up to one million gallons, and in all cases they have been completely free from leakage. When repainting of the steel has been necessary, considerable saving has resulted from the permanent nature of the floor.

Relationship Between Fire and Water Departments

By Jay W. Stevens

THE fire department and the water department are not, as some may suppose, distant cousins, but full blood brothers. When the fire chief of Pasadena was called out on an early morning alarm and discovered a large frame public garage on fire, with a 70-mile gale blowing, 3,500 trees blown down and 4,000 telephones out of order, he shouted, "Oh, Brother!" Brother Water Department was what he meant; and *how* he needed that brother. That fire was extinguished as the result of a very well organized fire fighting defense, backed up by—quoting the Chief's report—"a water supply that was adequate."

Over the years I had formed the impression that the fire department was the most neglected department of the city government; for, in the early days, there was little regulation and less cooperation on the part of the public. As I look back today, with a thought to the men in the water department, however, I am sure that, except for credit for providing water for drinking purposes and occasional baths, the water department was just as badly neglected.

An experience in my early days as battalion chief in the Portland Fire Department is illustrative of the difficulties involved in fire fighting at the time. Between 8:00 and 9:00 P.M., I received a telephone alarm of fire from the eastern city limits in my territory. I responded immediately, and with considerable concern for, as I left my quarters, I noticed that the sky in the vicinity was aglow. It made me very thankful that although an east wind was blowing, it was not the high east wind which we of the department had learned to fear. From the location I knew that we must depend upon buckets and garden hose as equipment; and I recall very distinctly that upon

A paper presented on May 10, 1940, at the Pacific Northwest Section Meeting, Portland, Ore., by Jay W. Stevens, Chief, Bureau of Fire Prevention, National Board of Fire Underwriters, San Francisco.

passing the last fire hydrant, two miles before we reached the scene of the fire, I said to my driver (this was in the horse-and-buggy days), "We can kiss the water supply goodbye. That's the last water we'll see unless by chance the fire comes to meet the water."

Upon arriving at the scene of the fire, I knew there was a serious possibility that the fire might extend into the city, for if the wind should rise to high proportions, with all of the frame residential area and its shingle roofs and no fire hydrants, the entire east side of Portland might easily be wiped out. This was a condition which we had feared for years and the fire was the nearest approach to a general conflagration of any during my entire experience, although I had responded to many other fires covering a greater area, and carrying with them a much greater loss. This fire was stopped by draping bed blankets and quilts over the roofs and sides of the small dwellings in the section, and by keeping them soaked with water from buckets and garden hose. The fire burned so rapidly that it burned itself out soon, before the intense heat had an opportunity to evaporate the water and ignite the blankets. It was the only time I have ever used this method, but I am convinced that the blankets and quilts, as a substitute for an adequate water supply, prevented a conflagration of major proportions. I am sure that that section of Portland, at the particular time, had potentialities far beyond conditions which caused the conflagrations in Chelsea in 1908, Paris, Tex., and Nashville in 1916, Atlanta in 1917, and Berkeley in 1923, and many others which have been attributed to wooden shingle roofs. The only mitigating feature in the conditions in Portland was the lack of high wind.

In my opinion, the general public has the cart before the horse as far as the water department is concerned. If I were building a water department for a city, my first throught would not be water for domestic consumption, not water for drinking or baths, nor for roses and lawns; these things are all very necessary, convenient and beautifying, but all are of no avail in a conflagration, and most of our American cities have at some time during their history suffered at least one conflagration.

When we speak of the San Francisco fire, we mean the 1906 fire, forgetting that in the earlier days a larger proportion of the city was destroyed on several occasions than was true in 1906. Every American city is confronted with the possibility of a conflagration, under the right conditions, every day of its life, and that thought

should be foremost in the minds of the city fathers when they build the water department, and as they continue to extend and improve it.

Water as an Extinguishing Agent

Let us for a moment discuss some of the conflagrations during previous years. While the statement of Franklin H. Wentworth, in his report of the Salem, Mass. fire, that conflagrations are never extinguished by water from hose streams is generally true, I am sure that water stopped the conflagrations in Nashville, Tenn., in 1916, where 648 buildings burned with a loss of one and one-half million dollars; in Fall River, Mass., in 1928, with 107 buildings burned and a loss of two and one-half million dollars; in Norfolk, Va., in 1931, where 60 buildings burned with a loss of one and one-quarter million dollars; and in Shreveport, La., in 1925, where 196 buildings were destroyed with a loss of one million dollars. A peculiar thing about the Shreveport fire was that it occurred while water mains were being repaired, cutting off the supply temporarily, and that it raged until the repairs were made, after which the fire was extinguished.

I am confident that in practically every conflagration, if the water supply is adequate, the equipment suitable for heavy high pressure streams, with sufficient trained man-power available, the fires can be controlled, although in some cases it would call for great strategy and probably take some time. The report on the Chicago stock yards fire bears this out.

In a number of conflagrations, deficient water supply contributed very definitely to the volume of the fire. This was especially true of San Francisco, where with all water cut off, and with 50 fires burning, a real catastrophe was inevitable. In the report of the conflagration at Augusta, Ga., in 1916, where 680 buildings were destroyed, causing a loss of four and one-half million dollars, the water pressures were reduced from a normal 60-pound static to 5 pounds, and when pumps were running to a 5-pound vacuum.

A fire department has one distinct advantage over a water department. When a fire department fights a conflagration, it is generally assisted by departments of neighboring cities, in some cases apparatus and men coming from a distance as great as 200 miles. When there is a shortage of electric power in one locality, power companies, through the use of emergency hook-ups, can divert power over unusual routes to the point needing reinforcement. But when a conflagration causes an unusual demand for water, each water sys-

tem must depend on itself alone; there is rarely any possibility of outside aid, or of switching water from remote points. Not only that, but the water must be available at the fire grounds, not in some reservoir where it can not be used, as was the case in the Berkeley conflagration of 1923.

Up to the time of the Baltimore conflagration, water superintendents were too prone to consider the question of water supply only from the standpoint of domestic consumption, giving little thought to fire-fighting. Unfortunately, this idea still persists to some extent, especially in cities of rapid growth, where realtors must have drinking water, if not fire protection, to sell lots. In the better governed cities, however, water department officials are fully awake to the necessity of providing sufficient water to extinguish fires. Usually there is the best of cooperation between the two departments. Sometimes the water superintendent and the fire chief make periodical inspection trips together, to check up on new buildings which have been erected, and to determine the best location for fire hydrants.

Number and Location of Hydrants

In passing I wish to emphasize the thought that there is no way by which suitable hydrant distribution can be secured except by personal inspection. To be sure, there are rules, based on linear spacing and on area supplied, which are of value under average conditions, but so many exceptional conditions are constantly arising that it is a wise precaution to see on the ground exactly what is required in the way of protection. A residential neighborhood, built up mostly with five-room bungalows on individual lots, does not require much water for fire protection, nor a particularly close spacing of hydrants. At intervals in such a neighborhood we may find occasional buildings of larger size, such as schools, churches and groups of stores. It is obvious that such structures need more water and additional hydrants for fire protection. As time passes, and the city grows, the detached dwellings may be replaced by apartment houses, or by industrial plants, requiring still larger quantities of water and closer spacing of hydrants. It is only by constant watchfulness that these changes can be detected and a high degree of protection maintained.

It is not uncommon to find cities economizing in the number of hydrants. Perhaps that is more true in the west and on the Pacific Coast than in the more closely built cities of the east. There is no

denying the fact that a hydrant and its installation costs money and if a large number are required, the total cost may be considerable. We must remember, however, that a hydrant has a long life, rendering effective service for fifty years or more. Under such conditions its *cost per year* is not large in comparison to the protection it affords. It is good practice to make a survey to determine the total number that should be installed, and to set a definite number each year, until all are in service.

Distribution System Records

There must be suitable maps of the water distribution system. In the old days maps were frequently in atlas form, each sheet showing mains, valves and hydrants in a limited area of the city. Although the individual sheets might be accurate, it was difficult to follow the course of the water from the pump or reservoir across sheet after sheet to the hydrant. As a result of this method of keeping records, weak places developed in pipe systems, undetected. In one large water system a nationally known boulevard had a six-inch main on both sides of the street, but as the boulevard formed the edge of the atlas sheets, the fact that there was no cross-connection between the mains for a distance of one and one-half miles escaped observation.

Another old-fashioned custom, which still persists in some of our cities, was to appoint the water superintendent for a two-year term, thus retiring him just as he was beginning to learn the duties of his office. Hand and glove with the custom, the out-going superintendent used to take all his records with him. As a result, the city had no map to show, nor any information to give a conscientious fire chief seeking to learn something of the water system upon which the success of his own work depended. Fortunately, such cases are becoming more and more infrequent with the passing of time.

There are almost as many ideas about maps as there are engineers. From the point of view of the fire chief, there should be a map to show mains, gate valves, fire hydrants, pressure regulators, limits of pressure zone, pumping stations, reservoirs and elevated tanks. It adds to the convenience to have plenty of street names, besides points of special interest, such as parks, schools, railroads and grade separations, water courses and bridges. Four-inch pipe should be shown by a special symbol, as localities served only by pipe of this size are likely to be weak from the fire protection point of view.

Where large feeder mains are not connected to smaller mains on which hydrants are set, the fact should be so indicated. An excellent example of what the fire chief would like to have is that recommended by the Association's Committee on Distribution System Records (Jour. A. W. W. A., **32**: 188 (1940)).

If possible, the map should be large enough so hydrants can be tinted to show their delivery. The map should be made on tracing cloth so that it can be easily brought up to date and new prints made as required by changes in the system. Where the area of the city does not exceed twelve or fifteen square miles, and a scale of 600 ft. to the inch is selected, the map can be made in one sheet, but if the city is very large, two or more sheets may be necessary, if clarity is to be preserved. This may sound like a lot of work, but if you could go with me to visit some fire stations, see for yourselves how carefully firemen have mounted such maps, and note the evidences of frequent study, you would agree with me that it is useful work, and that the time has been well spent.

Fire chiefs derive much benefit from a study of the reports on fire fighting facilities prepared and issued by various underwriting organizations. I assume water works men give them equal consideration as the water supply section is a carefully compiled and impartial discussion of the strong and weak features of the water system, based on the possibility of the spread of a sweeping fire. In fact, the Baltimore conflagration of 1904 was the immediate cause of the entry of the National Board of Fire Underwriters into municipal affairs on a large scale. During the fourteen years previous to that date, a large number of reports on cities had been prepared by single inspectors, usually fire chiefs of long experience, and, as was natural from their training, little space was devoted to water works. The Baltimore fire caused a direct property loss of 50 million dollars, and aroused the insurance companies to the possibility of similar fires in other cities.

This conflagration was carefully studied from many angles and the preparation of reports on all the large cities of the United States was begun in a systematic way. Instead of single inspectors, the work was carried on by parties composed of engineers specially trained in the branch of the work they were to handle. In addition to description and criticism of water systems, the reports outlined fire departments, fire alarm systems, actual structural conditions and laws and ordinances relating to building and fire hazards—all leading

up to a discussion of the possibility and probability of a conflagration—followed by recommendations for improving conditions thought to be unsatisfactory.

The National Board of Fire Underwriters published its first report, on the city of San Francisco, in October, 1905. This report discussed systematically all the features which had a bearing on the fire defenses of the city. The last page is devoted to a summary of the conditions found. The last two sentences read as follows: "In fact San Francisco has violated all underwriters' traditions and precedents by not burning up. That it has not done so is largely due to the vigilance of the fire department, which cannot be relied upon indefinitely to stave off the inevitable." The very last word in the report is *inevitable*. You all know what happened the following April.

It is the general rule of the National Board of Fire Underwriters to issue reports only on cities of 25,000 or larger population. Up to the present time some 450 cities have been reported on. In cases where the city has barely passed the 25,000 population mark only one report has been prepared, but for some of the older and larger cities as many as six or eight reports have been issued.

Relative Importance of Water

Since the World War, every report made by the National Board of Fire Underwriters has been accompanied by a grading to show the relative classification of the fire defenses. This is simply a mathematical expression of the efficiency of the various factors which bear on the possibility of a sweeping fire. This method of grading has been adopted by all the fire insurance rating organizations on the Pacific Coast and is applied in exactly the same way. Every city, both large and small, is judged by the same yardstick. Not only can the efficiency of existing conditions be ascertained, but the effect of proposed improvements on the grading can be estimated so that any city can know in advance whether or not the project is economically feasible.

Shortly after the introduction of the grading schedule, and before its operations were thoroughly understood, some fire chiefs were disposed to look askance at the relative weights assigned the various features. Although the schedule for years has been printed as an appendix to the A. W. W. A. *Manual*,* I may venture to remind you

* *Water Works Practice—A Manual.* American Water Works Association, New York, 1926.

that of the 5,000 deficiency points, 1,700 are chargeable to the water department, and only 1,500 to the fire department. The carping fire chiefs felt that, as they were more active in extinguishing fires, they should be allotted the greater number of points. The fact remains, however, that water is the fundamental basis of all major fire fighting, has been so since the days of the Romans, and, in spite of recent inventions, will continue to be so for years to come. A water supply without a fire department may be of some value, but a fire department without a water supply is helpless.

For all practical purposes, both departments are indispensable. Ordinarily, the work of the water department in providing sources of supply, laying mains and setting hydrants, is completed before the fire starts, and the actual extinguishing of the fire is left entirely to the fire department. This procedure works out well in the great majority of cases, but occasionally there are large fires where the water department can be of service at the scene of the fire. It is not uncommon for large service connections—three, four, six or even eight inches in diameter—to be broken off during a large fire and for the escaping water to reduce pressures in the mains so that a normally abundant fire supply may become insufficient. The Salem conflagration, with the breaking of eight-inch connections, is a classic illustration of the disastrous consequences that may follow. To stop the flow of water being wasted, is a job that should be done only by an experienced water works man.

Where a city is hilly and water is distributed in several pressure zones, it is sometimes feasible to increase the supply at a given point by opening emergency valves from a higher zone. This operation is one which requires a thorough knowledge of the entire water system, in addition to experience and good judgment. Such situations do not develop frequently and it would be asking too much to request that the water department send representatives to all fires. It is, however, fast becoming good practice for water departments to send one or two capable employees to all second alarm fires and to first alarms in the central business district. The number of such alarms during the course of a year is not great. The responding crew should be provided with an emergency truck equipped with necessary tools and records, preferably in the form recommended. Where no response to fires is made by the water department, the failure may be the fault of the fire department, which may not have developed an effective plan for transmitting alarms of fire to the

water department. In such a case, a conference between the heads of the two departments can usually work out a satisfactory procedure.

Chicago Stock Yards Fire

To me, the underwriters' report on the Chicago stock yard fire of May 19, 1934, is more thrilling than any work of fiction. The picture of the fire department making stand after stand, being driven back by heat too great for any human being to endure, and finally gaining the victory, is unequalled in fire protection literature. Handicapped at the start by the absence of city mains and the scarcity of hydrants, the fire department was able to extinguish the conflagration only by using water at the unprecedented rate of 50,000 gallons per minute, a higher rate than has ever occurred in the city of Portland. That the fire was finally stopped attests the strength and efficiency of the fire department, the long hours devoted to drills and training, the use of the most modern fire methods, and the perfectly maintained pumpers; but it must not be forgotten that without the immense volume of water provided by the water department, there is no telling how far the fire might have traveled. Instead of a loss estimated at $5,500,000, the loss might have been a hundred times as large, or even more.

The water department officials quickly realized the seriousness and the possibilities of the fire. Due to the hot, dry weather, and the occurrence of the fire at the time of day when lawn sprinkling produced maximum consumption, it was promptly realized that in order to provide adequate quantities of water at sufficient pressure, and to supply ordinary consumption demands, lawn sprinkling would have to be curtailed. Requests were therefore made to radio stations to make appeals to consumers to shut off their lawn sprinklers and otherwise to conserve water. These requests were first broadcast forty minutes after the fire started, were repeated at short intervals, and were so effective as to obviate the necessity for other emergency measures. Because both the water and fire departments were prepared to meet the crisis, Chicago can be thankful that the year 1934 was not a repetition of 1871.

In recent years, fire chiefs—and I am sure this is also true of water department heads—have come to realize more and more the value of close cooperation between these two departments. Each can do much to help the other. If either is permitted to drop to a low stage of efficiency, the safety of the community is jeopardized.

The Rehabilitation of Water Mains

By Earle S. Hoyt

EARLY in 1938, the City of Marietta undertook a complete survey of its water works system with the view to modernizing and correcting deficiencies. The decision to make this investigation climaxed a period of several years during which much criticism had been leveled at the water facilities in Marietta. The local people were dissatisfied with the water; the quality of the supply from a sanitary viewpoint had been condemned by the State authorities; the Ohio Inspection Bureau had reported inadequate fire flows, and excessive pressure losses at several points in the system; and there was evidence of deterioration of some structures.

The first municipal water supply system at Marietta was built in 1890–91. It consisted of a dug well on the bank of the Ohio river, a pump station, two steel storage tanks and about 12 miles of mains. By 1898, the system had expanded to include about 24 miles of mains. At the time of the investigation there were a little more than 40 miles of mains in the system. This, of course, indicates that the age of more than half of the distribution mains was at least 40 years.

The original water supply was not satisfactory because of its high iron content, and almost as soon as the first plant was built, arrangements were changed so that the supply was taken from the Ohio River. During the first twelve years of operation, other sources of supply were sought, principally from wells along the Ohio but none was found satisfactory. In 1905, the city constructed a filter plant which was used with practically no changes until last year. The water treatment consisted generally of coagulation with lime and copperas or alum. For several recent years the use of alum or iron sulfate had been replaced by mixing with the river water a portion of the high iron-bearing water from the original well.

A paper presented on May 10, 1940, at the Ohio Section Meeting, Akron, Ohio, by Earle S. Hoyt, Sanitary Engineer, Columbus, Ohio.

The general layout of distribution mains and storage at Marietta was good, static pressures in congested areas were from 85 to 100 pounds but fire flows were much lower than pressures and pipe sizes indicated. There was much talk among the local authorities of the need for increasing pressures—a remedy that the layman is quite likely to think of first when flows are deficient. Among the local people, the effect of interior condition of the mains was not generally recognized except in a few locations where the trouble was attributed to some poor pipe purchased before the adoption of present day manufacturing standards. Samples cut from mains in various parts of the system, however, showed a heavy tuberculation. Flow tests on various isolated stretches showed carrying capacities reduced by 50 per cent or more.

Tuberculated mains alone, however, were not the sole cause for the deficient fire flows at Marietta. Other contributing causes were found to rest on poor records and lack of proper valve inspections. Two closed valves were found in the city and one other about half closed. One of these valves had been completely lost. There were no good maps of the distribution system—the one map available was known to be incomplete and inaccurate. The system contained both right- and left-hand valves. In the early 1920's, a 12-inch feeder to a particular industrial development was laid around two sides of the main grid system and another 12-inch feeder around a third side of the grid, but these feeders were not generally connected into the grid system. In other words, only at a few widely separated points were they connected to the grid system although it was found that crosses and valves had been installed in the mains at many intersections. These installations had been buried and lost and, of course, had never been connected to the grid.

Rehabilitation Program

As the result of the investigation, the city projected a general rehabilitation program which included the following features:

1. Development of a new source of supply consisting of ground water from the gravel terraces along the Muskingum River, with the river itself as a standby source.

2. Construction of a modern water treatment plant including softening and stabilization equipment.

3. Repair of existing storage facilities.

4. The cleaning of nearly 90 per cent of all the cast-iron mains—some 178,675 ft.

5. A general reinforcement of the distribution system by completing the feeder loop around the main grid and tying in the 12-inch feeder on all sides of the grid at most of the street intersections. The distribution was further reinforced by the addition of two equalizing reservoirs on the ends of long outlying mains up and down the Ohio River.

6. The extension of service to a high level area by a booster system.

Contracts for the new supply, the treatment plant, the additional distribution system mains and the reservoir work were let late in 1938. Awarding of the contract for cleaning mains was purposely postponed until the early summer of 1939, so that the main cleaning would not be completed too long before the time when the new water supply would serve the city. This was done because among samples cut from mains, one from pipe which had been taken up, cleaned and relaid four years previous to sampling was found to be as heavily tuberculated as samples from sixteen-year-old mains. Also, one sample in place only twenty months was found to be well started toward tuberculation. The main cleaning work was begun about the first of June and completed in November.

Main Cleaning

The work was handled by contract as a P.W.A. project and all of the costs were borne by the contractor, i.e. all equipment, labor and material was furnished under the unit prices bid. New material added in the form of valves and pipe was at unit prices. The total contract cost for cleaning 34 miles of mains amounted to an average of a little over $500 per mile.

Table 1 shows some of the typical results of fire flow tests run before and after the main cleaning and other distribution main improvements. These tests indicate that the main cleaning alone increased fire flows by approximately 50 per cent with much better residual pressures and that the entire program increased fire flows from 135 to 180 per cent with better residuals.

The main cleaning contract included a provision that mains were to be restored to a carrying capacity equivalent to 95 per cent of capacity, computed with a Williams and Hazen coefficient of 110 for 4- and 6-inch and 120 for 8-inch and larger pipes. Tests indicated that this capacity was reached or bettered.

New Cleaning Machine

One of the interesting features in connection with this main cleaning program was the use of a new type of cleaning machine, particularly applicable to the smaller pipe sizes, 4- and 6-inch, which enabled the contractor to accomplish his work with fewer cuts in the line and with greater rapidity. This machine, instead of using the scraping mechanism which is drawn through the mains by cable or pushed through by pressure, used a rotating head driven through flexible rods by a gasoline engine mounted on a small pneumatic-tired chassis. A section of pipe about 36 in. long was cut from the main and the revolving head inserted in the open end of the main. This revolving head consisted of a center shaft with guide collars for centering the shaft in the pipe. Attached to the shaft were short chains on the ends of which were specially hardened cast-iron blocks, so that when the shaft was revolved, the head became a sort of hammer-mill. The flexible rods led from this center shaft up to the small power-driven machine which was pushed or power-driven along the pavement. About 48 ft. of flexible rods were connected to the machine at a time and the revolving head advanced into the pipe by

TABLE 1

Results of Fire Flow Tests

CONDITIONS	FLOW	STATIC PRESSURE	RESIDUAL PRESSURE
	g.p.m.	*lb.*	*lb.*
10-inch Line			
All valves on intersecting lines closed:			
Before cleaning	1,120	83	34
After cleaning	1,700	90	65
All valves on intersecting lines open:			
Before cleaning	1,660	80	27
After cleaning (with grid improvements)	3,900	83	55
12-inch Line			
All valves on intersecting lines closed:			
Before cleaning	1,190	83	26
After cleaning	1,835	94	50
All valves on intersecting lines open:			
Before cleaning	1,260	81	25
After cleaning (with grid improvements)	3,600	89	42

the movement of the driving machine on the pavement. The machine was then disconnected, moved back, and another 48-foot length of rod attached; and so on up to lengths in excess of 1,200 ft. During the time the machine was advancing into the pipe, a small amount of water was permitted to flow from the open end of the main, and when the cleaning head reached a valve, the valve was opened and the cleaning head went right on through it up to the intersecting main, or as far as desired. When the end of a section to be cleaned was reached, the machine was withdrawn by reversing the general procedure, except that a larger supply of water was furnished to remove the debris from the pipe. The main was then thoroughly flushed, and, where locations warranted, the machine could from the same cut be turned and run 1,200 ft. or more in the opposite direction.

Summary

Marietta's experience emphasizes again several important facts:

1. The carrying capacity of well constructed distribution mains can be restored very nearly to original values at nominal costs.

2. The keeping of accurate maps and records is essential to the proper maintenance of a distribution system.

3. The use of both right- and left-hand valves in the same system should be avoided.

4. In determining the treatment to be applied to any water supply, consideration should be given to the action of the treated water on the distribution system.

Attention is called to the fact that the accumulation of "deferred maintenance" to the extent reported here can be avoided by *competent management.*

Ownership and Maintenance of Service Lines and Curb Boxes

By William C. Shoemaker

IT HAS been decided in recent years by the higher courts that where a municipality or a water company pays a portion or all of the cost and expense of installing a service pipe, or other accessories, and the property owner pays the balance of the expense, the company is the legal owner of the service line or accessories. Under these circumstances, the municipality or water company is obligated to make frequent inspections of the service lines and boxes to be certain that there are no leaks which might result in injury or damage to persons or property, and that the visible parts of the service, such as meter box tops or curb box tops are at the proper grade and do not represent a hazard to the public.

Only where the entire service costs are borne by the property owner and where no further maintenance agreements are in effect, requiring the water company or municipality to maintain services after installation, will the utility be free of the obligation to inspect and keep them in good and safe condition.

In this respect some Indiana plants seem to have epidemics of accidents, some minor and some very serious, as a result of injuries sustained by tripping over meter box or curb box tops. These box tops are very difficult to keep at the proper grade. Erosion, digging up of the box, cutting down of curb grades, all mean that the box may be left above the ground level to constitute a hazard. A box even a fraction of an inch high, with grass around it, means that some person may accidentally trip over it and sustain a serious injury.

Some of the larger plants have had to add special inspectors to their payrolls—men whose duty it is to inspect curb boxes and

A paper presented on April 4, 1940, at the Indiana Section Meeting, Lafayette, Ind., by William C. Shoemaker, Manager, Richmond Water Works Corp., Richmond, Ind.

meter boxes, and promptly report those in need of attention. For smaller plants, and those who have had little difficulty with their boxes, it seems that the meter reader could be given the job of checking all boxes and marking his observations on the meter sheets, most of which are provided with a code for such purposes. This would not take much of the reader's time, and as he is the only person in the organization who regularly goes over every street, either monthly or quarterly, in most instances he could be given that extra duty.

But regardless of whether a plant is large or small, whether there have been many such accidents in the past or not, box lids will always represent a hazard to the public, and as such, a liability to the plant. It will be well to caution street department employees as to the necessary care to be taken in placing the boxes, to ask them to be on the watch for boxes that are dangerous, and to have someone, either meter readers or full-time inspectors, to be always on the look-out for equipment that may represent a hazard to the public.

Maintenance

There is little that one operator or superintendent can tell another in regard to the maintenance of services or boxes. Each locality has problems peculiar to its own conditions. Depth of mains, types of pavements and streets, various service materials in use and local rules and regulations vary with each community. But considering maintenance in the light of replacement of services, there are a number of ideas which it may be valuable to review.

The winters of 1935 and 1939, have demonstrated to a number of superintendents the necessity of providing more cover over their pipes. This, on a large scale, is not practical, but such conditions should be kept in mind when replacing services.

When a service must be renewed completely it might just as well be installed properly, that is, below the lowest frost line. Sometimes, with shallow mains, it means making a new tap in the side of the main, so that the service can be lowered without having a gooseneck or pipe bend extending six inches or a foot above the main. In the majority of plants, the forty-five degree tap is preferable, but in order to lower the service connection the ninety-degree, or side, tap should be considered.

In this connection, it may be pointed out to those who are using cement-lined cast-iron pipe, that it has recently come to our atten-

tion that the ordinary drill and tap, as has been supplied in the past, will not hold up when tapping cement-lined pipe. The cement lining will ruin the relief on the threads of the tap, sometimes within the first few trials. There are special drill and tap sets now supplied that will overcome this difficulty.

Type of Service Material

Type of service material is a subject that has been more thoroughly discussed by water works men than any other one topic. There are literally scores of references, in the water works literature, to discussions, investigations and reports upon the subject of proper materials for service pipes, but we still find all types of materials in common use. For the proper materials there are certain simple requirements: First, the material should not be such as to impart to the water, qualities injurious to the health of the consumer; or to change the appearance, taste or odor; or to increase the hardness, iron, copper or zinc content of the water. Pipe material should be of such a character as to permit the pipe to retain its carrying capacity; to be strong and durable under the conditions of service; to be easily laid with simple tools and a minimum of skilled labor; and to be relatively inexpensive, taking into consideration the probable life.

We have in use today many kinds of service materials—lead, steel, wrought iron, brass, copper and cast iron. Lead, one of the earliest commonly used materials in this country, has the advantage of flexibility, ease of laying and durability. It has the disadvantage of relative high cost, need for skilled workmen and the danger of lead poisoning with certain waters.

Steel and wrought iron have the advantage of simplicity of installation, rigidity for jacking, fair durability and low cost. But even when galvanized, they are subject to corrosion, inside and out, and frequently produce rust in the water passing through them.

Brass is rarely used because of the high cost of the material.

Copper tubing has become quite popular in the past fifteen years, more so since it has been available in coils. It has the advantages of flexibility, is relatively non-corrosive, simple to install and not too expensive. It has a disadvantage of being subject to corrosion with soft ground waters carrying carbon dioxide.

Cast iron has become more popular in the past few years in sizes from $1\frac{1}{4}$ inches and larger, and mostly in the 2-inch size.

After reviewing the experience of water works operators, we can

only say that there is no uniformly satisfactory material now in common use.

For services of $\frac{3}{4}$ in. and 1 in., copper is preferred at Richmond. It is flexible, easily laid, requires a minimum number of tools, no skilled labor, withstands freezing better than other materials, and is not high in cost. In general it will cost about 15 per cent more than galvanized iron, 30 to 40 per cent cheaper than lead and 30 per cent cheaper than brass.

For those services larger than 1 in., cast iron is used, usually in the 2-inch size, cement lined. This service material can be had either with precalked lead joints or with iron pipe threads. The threaded 2-inch pipe is used with $2\frac{1}{2}$-inch iron pipe fittings and valves and now is furnished in 9-foot lengths. For ordinary services no expansion joints are necessary. Connections to the main are made with two, three or four 1-inch corporations, depending upon the quantity of water required. Copper is used from the corporations to the iron pipe branch. It is felt that if a customer needs more water than can be furnished with a 1-inch copper service that a permanent installation of 2-inch cast iron should be made to insure the best of service and to provide for any additional requirements in the future. Cast iron is not much more costly than the larger sizes of copper.

Curb Stops and Boxes

In the choice of curb stops, we are limited to two patterns. Either the solid tee head with the tightening nut on the bottom, or the inverted key. While both are in use today, the inverted key is more generally used. It is slightly higher in cost, but the working parts are entirely enclosed, better seating of the core is obtained by having water pressure at the bottom, and the core can be loosened by tapping the tee head slightly, enabling the stop to work easily after many years in service.

Curb stop boxes may be had in a variety of styles and sizes and several different types of covers or caps. The larger diameter boxes, that is $1\frac{1}{2}$ in. and 2 in. in diameter, are used by having the valve key inserted all the way down to the curb stop. A box, the upper part of which is only 1 in. in diameter, with a stationary rod fastened to the curb stop head, and the rod extending to within about a foot of the ground level is much preferred. This requires only a short key for operation, and does away with dirt accumulating around the curb stop and making it difficult to locate with a long key. There

are a number of so-called locks or patented devices for locking curb box caps, but the most satisfactory is a screw cap, the female thread being of a brass bushing, and operated either with a spanner wrench or a wrench fitting into a recessed pentagon head.

In closing, it might be added that, coming under the heading of maintenance of service pipes, there is another not-to-be-forgotten item that is a cause of worry and grief to many superintendents. That is the abandoned service. Too often services are abandoned, or replaced with larger ones, and are left on at the main, either because the service crew is too busy at the time to shut it off or because there is a hope that at sometime that service may be used again. But those abandoned and oft-times forgotten services may be the cause of an enormous waste of water and may leak over a long period of time before they are finally discovered. The time to discontinue an old service is when it is replaced with a new one or when it is definitely decided to abandon it. It should be cut off at the main and the service map marked accordingly. This will save considerable loss and unnecessary expense in later years.

Increasing Water Sales

By D. W. Robinson

RECORDS of water use over a long period of time show a slow but steady increase in water sales as a result of a higher living standard. In most water systems, however, there are also numerous other opportunities to increase sales.

Most water systems, especially the smaller ones, have surplus capacity required for fire protection and through this added capacity can increase their sales materially without further expenditures for supply. Any sales over and above the bare production cost will mean more money left over for bond interest, extensions and improvements—and perhaps a little for better salaries.

The figure, one hundred gallons per person per day, is usually considered average usage. Some systems show a consumption up to three times this amount—lots of room for increased sales.

Increasing sales by means of selling appliances is not practicable for most water systems, even though it works out very satisfactorily for gas and electric systems.

While a water works system is usually considered a monopoly, there are very few which do not face competition from some source. There is only one successful way to meet competition and that is by convincing the customers that your company can give them equally good or better service for less money.

It is usually too much to expect immediate results from any efforts toward increasing sales. The process is one of education and therefore slow.

The two main sources of increased sales are, of course, increased sales to present customers and sales to new customers.

Sales to present customers can sometimes be increased materially

A paper presented on October 14, 1940, at the Southwest Section Meeting, Tulsa, Okla., by D. W. Robinson, Engineer, Community Public Service Co., Fort Worth, Texas.

by rate revisions and by this is not necessarily meant rate reductions. Many forms of rates offer the customer no inducement to use more water. We shall not here discuss such rates in detail or the changes possible, but no revision will ever be satisfactory unless all costs are known; and a satisfactory accounting system is necessary to such a knowledge.

A study of costs will show that only some will be increased by increased sales. Any rate which is more than enough to cover these increased costs will leave more money to carry the other indirect overhead costs. A correct knowledge of costs is, of course, necessary to avoid setting rates for large consumers below the cost of production.

Irrigation Rates

Irrigation rates can often be made a source of increased sales. The Community Public Service Co. of Fort Worth, Texas, has irrigation rates in effect in many towns and finds a marked increase in sales. The rates consist of an increased minimum bill allowing an increased amount of water to help pay for the increased demand and then a low rate for all excess. In some of the drier parts of the state, domestic customers, living in homes only a little above the average, have bills as large as $25 to $30 per month during the summer.

Closely related to the establishment of irrigation rates is the active cooperation of the water utility with luncheon clubs, womens clubs, garden clubs, etc., in civic beautification. Prize contests can often be developed to stimulate pride in home yards, lawns and gardens. This will materially increase water sales and will also make your city a better place in which to live.

Most water systems now believe in landscaping the grounds around their plants. By experimenting with different shrubs and flowers, the water department may find those best suited to the locality and serve as an adviser on plantings.

Golf courses, parks, airports and cemeteries are good prospects for water for irrigation. One inch or more of water per week is usually considered to be necessary to maintain good turf. This may come from rain or sprinkling. Watered greens and fairways are becoming a necessity for golf courses in order to meet competition. An average golf course will have 50 acres of greens and fairways requiring watering. If such places are municipally owned,

however, it will, of course, be necessary to be sure that the water system will secure payment for its service.

If a large investment is required to serve similar businesses which are privately or semi-privately owned, an investigation of their finances should be made. Many such organizations are in financial troubles frequently and may not be able to pay for water.

The above types of customers are particularly desirable because most of their watering is done at night in order to prevent interference with normal use of the grounds.

The proper selection, sizing and maintenance of meters may aid in increasing sales. The amount of revenue to be obtained by such increased sales may not increase materially unless the rate is properly prepared.

Increasing pressure to the consumers, either constantly or periodically at peak periods, will materially increase sales. Provision of adequate elevated storage, booster pumps and proper sized mains and services will result in more consumption. The poor service conditions which some people will tolerate because they have developed so slowly that no one realizes just how bad the service has become are surprising.

Replacement of Auxiliary Supplies

Many customers have an auxiliary water supply used for some purposes. A study of their needs may enable the department to show them how they may save by using the city supply for all their needs. Cost is not always the determining factor since quality may be important as well.

The maintenance of an auxiliary supply also usually requires maintenance of duplicate equipment, extra labor and many overhead charges, and this can often be taken as a point of departure.

When a customer is faced with extensive repairs, improvements or changes in equipment, the time is opportune to try to get him to abandon his auxiliary supply. Railroads, factories, ice plants, power plants, bottling works and other plants requiring large amounts of water are to be considered in such a drive.

Under certain conditions it may prove advantageous to attempt to promote additional sales by supplying water for air-conditioning; and, providing a satisfactory rate can be secured, fire sprinkler installations are also a possible source of revenue.

Swimming pools are good water customers. The non-circulat-

ing type will use more water but will usually require a large line for frequent fillings in a short time and may seriously overtax a small system. The circulating type pools are preferable from a health standpoint. Since they are not refilled so frequently it is usually possible to serve them with a smaller line. They do require considerable water between fillings, however, for makeup, showers, toilets and filter washing. A swimming pool located near the residential section is of course better for the water system and is more likely to be financially successful.

New customers can be obtained from those people who are now using private wells. Private wells are often a health hazard and should be discouraged. A prolonged dry spell is usually a good time to secure such customers for their wells often fail during such periods. If the locations of such private wells are known, it is easier to solicit their business during a period of dry weather.

Many times a short extension of small pipe will secure several customers and if a supply from the city system is available, it will prevent new house owners from digging wells. A small line will hold such business until the territory develops sufficiently to justify a larger line. In this respect, knowledge of the trends of growth of the city will be of value in determining the probability of future growth.

Water can often be sold wholesale to neighboring towns or suburban areas. This is sometimes a source of considerable trouble, so it is probably better if the utility can serve them directly under some sort of extension agreement. It is best, too, to have a standard policy worked out to cover such service. Care must be taken in making such agreements to avoid discrimination between such areas and others in the city.

To sum up, the requirements of building water sales are: a proper rate, a complete knowledge of production costs, a study of available competitive sources, a knowledge of the present and prospective customers and their needs, and last, and most important, a lot of hard work.

The old saying that it is not possible to stand still applies to water systems as well as to men; and if they are not growing they are usually going backward. The experiences of other utilities show that load-building efforts pay dividends in the end and are well worth while.

Ground Water for Public Supply in Western Pennsylvania

By R. M. Leggette

GROUND water has long been utilized as a satisfactory source for public water supplies in many parts of the United States, particularly in the smaller or moderate size communities. It has been estimated that there are about 6,500 communities (20,000,000 population) with public water supplies derived from wells (1). Fifteen of these communities are cities having a population between 100,000 and 300,000. The largest development of ground water for public supply is on western Long Island, where the average withdrawal during the period, 1904 to 1939, was about 136,800,000 gallons a day. The average daily draft during 1916, the peak year, was about 189,000,000 gallons. It is estimated that in 1930 more than $1\frac{1}{2}$ million people in New York City and suburban Long Island were dependent on public supplies derived from ground water sources.

Many parts of the United States experienced more or less severe droughts during the past decade with the result that some communities were faced with a water shortage. In general, the public water supplies derived from wells were adequate during these dry periods. Shortage of water occurred mostly in those communities utilizing surface water with inadequate storage facilities.

Public water supplies derived from ground water throughout the nation obtain water from a variety of formations, ranging in geologic age from the old Pre-Cambrian rocks to the Recent alluvial deposits that occur in present day stream valleys. For the country as a whole, the most important water-bearing formations are the deposits of unconsolidated sand and gravel of relatively young geologic age.

A paper presented on September 18, 1940, at the Western Pennsylvania Section Meeting, New Castle, Pa., by R. M. Leggette, Geologist, U. S. Geological Survey, Jamaica, N. Y.; and published by permission of the Director, U. S. Geological Survey, U. S. Department of Interior.

Survey of Resources in Pennsylvania

Pennsylvania was one of the first states to recognize the need for a systematic and state-wide survey of its ground water resources. In 1925, the Geological Survey of the United States Department of Interior, in cooperation with the Pennsylvania Topographic and Geologic Survey began a state-wide investigation of the ground-water resources of Pennsylvania. The first field work was carried on in the southeastern part of the state. Similar field work was done in other parts of the state in later years. The results of these investigations were published in six separate reports of the Pennsylvania

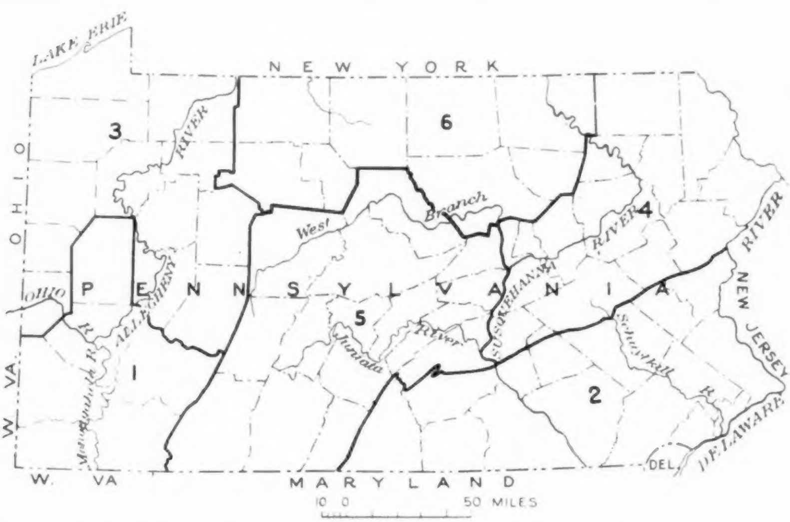

FIG. 1. Index Map of Pennsylvania; showing areas covered by ground water reports. Area 1, covered by Bulletin W1, Area 2, Bulletin W2, etc.

Topographic and Geologic Survey as Bulletins W1, W2, W3, W4, W5, and W6, each of which covers a group of counties (Fig. 1).

In this paper, western Pennsylvania is considered to include the 18 counties covered by Bulletins W1 and W3, an area of about 13,000 square miles. For more detailed information than can be given in this paper the reader is referred to these two publications (2, 3).

It is estimated that more than half of the public water supply systems in western Pennsylvania derive their supply from ground water sources. The population dependent on public supplies derived from wells and springs was about 275,000 in 1930.

Geology of Western Pennsylvania

The water-bearing formations in western Pennsylvania range from highly permeable deposits of sand and gravel to dense, compact shales of relatively low permeability. The formations range in geologic age from Recent alluvium to sandy shales of the Upper Devonian series. The formations are generally variable in both composition and water-bearing properties.

The surface of most of western Pennsylvania consists of either residual soil or of glacial deposits. During the glacial epoch, ice sheets covered the northwestern part of the state. The glacial boundary extends from the vicinity of Warren southwestward to the Ohio State line, a few miles north of the Ohio River. Northwest of this line the surface is made up of glacial till, consisting mostly of clay and boulders, or outwash, consisting mostly of sand and gravel. Many of the valleys in the glaciated area are filled with glacial outwash sand and gravel, more than 400 ft. thick in places. These deposits were laid down by streams formed by the melting of the glaciers. Similar deposits of somewhat less thickness occur in some of the valleys beyond the glacial boundary. Glacial outwash sand and gravel constitute one of the most important sources of ground water in western Pennsylvania. Properly constructed wells in favorable locations yield more than 1,000 g.p.m. in parts of the area. A map showing the location of the buried valleys is included in Bulletin W3. The waters from unconsolidated sand and gravel generally contain moderate amounts of dissolved mineral matter. They are commonly moderately hard and in places may contain objectional amounts of dissolved iron.

The oldest bedrock in the northwestern part of the state occurs chiefly in the northern area, whereas the youngest bedrock occurs in the southwestern part of the state. Each of the formations will be described only briefly in this paper in the order from youngest to oldest. Detailed descriptions and geologic maps of western Pennsylvania are included in Bulletins W1 and W3.

The Greene formation occurs chiefly in Greene and southern Washington Counties, where it has a maximum thickness of about 725 ft. It is composed mainly of shale and shaly sandstone with a few thin limestones. The sandstones, where coarse-grained, yield moderately large supplies of water. The shales generally yield small supplies where they lie near the surface. The waters have moderate amounts

of dissolved mineral matter, usually hard at shallow depths but softer at greater depths. Some of the waters are high in dissolved iron.

The Washington formation, underlying the Greene formation, ranges in thickness from about 275 to 440 ft. It crops out chiefly in Washington and eastern Greene Counties. The formation consists of sandy shale and sandstone with some limestone and coal. A sandstone member near the base of the formation yields as much as 65 g.p.m. to drilled wells. The rest of the formation generally yields small supplies at shallow depths. The water from this formation is usually of good quality with moderate amounts of dissolved mineral matter.

The Monongahela formation underlies the Washington formation. Its thickness ranges from about 260 to 400 ft. The formation is at or near the surface chiefly in northern and eastern Washington, southwestern Westmoreland, and southern Allegheny Counties. The formation is made up of shale, sandstone, thick limestones, and workable beds of coal. The limestones yield as much as 25 g.p.m. to drilled wells of moderate depth. The sandstones yield small to moderately large supplies of water. A sandstone near the base of the formation is coarse-grained in many places and is therefore quite permeable. Mining of the Pittsburgh coal that underlies this sandstone has, however, drained the permeable beds above, and in these areas the sandstone is no longer a source of water. The waters from shallow depths are generally hard, whereas those from greater depths are softer, though highly mineralized in places.

The Conemaugh formation, which lies next below the Monongahela formation, has a thickness of about 500 to 750 ft. It crops out chiefly in Beaver, southern Butler, northern Allegheny, northern Westmoreland, Armstrong, and Indiana Counties. The formation is predominately shaly but as many as five massive sandstones occur within the formation. Over considerable areas, these sandstones are good water-bearing beds that yield as much as 100 g.p.m. to drilled wells. The water-bearing properties of these sandstones, however, vary considerably from place to place. The concentration of dissolved minerals in the waters from the Conemaugh formation ranges from large to small amounts. The sandstones generally yield a water moderately high in dissolved iron.

The Allegheny formation underlies the Conemaugh formation. Its thickness ranges from about 250 to 370 ft. The formation crops out chiefly in Lawrence, northern Butler, northern Armstrong,

Clarion, and Jefferson Counties. The individual members of the formation, consisting of shales, sandstones, and coals, are quite irregular in both character and thickness. In places, the sandstone members are massive and coarse-grained and yield as much as 300 g.p.m. to drilled wells. In relatively short distances the sandstones may grade into shale that yield little or no water. The formation generally yields moderately mineralized water, that in places is quite hard. The water from the sandstones is usually high in iron.

The Pottsville formation, underlying the Allegheny formation, is the lowest formation of the Pennsylvanian series in western Pennsylvania. It has a maximum thickness in this area of about 275 ft. The formation crops out chiefly in Mercer, Venango, and Forest Counties. In most places, it consists of massive sandstone or conglomerate near the top and bottom with shaly beds in the middle. The sandstones are fairly persistent in occurrence as compared to the sandstones in the overlying formations. In many places, very good supplies of water are obtained from the sandstones, although the water is usually high in iron.

In the southern part of western Pennsylvania the Mauch Chunk formation underlies the Pottsville formation, but farther north it is absent. The formation, which consists of shale and limestone, is not important as a source of water in the area.

In the northern part of the area, the Pottsville formation lies on the eroded surface of the Pocono group. The Pocono has a thickness of perhaps 650 ft. It consists of sandstone at the top and bottom with shaly beds in the middle. These rocks are at or near the surface chiefly in Crawford and western Warren Counties. The sandstones yield good supplies of somewhat mineralized water at moderate depths, but where deeply buried, they generally yield highly concentrated water.

The oldest rocks in western Pennsylvania crop out chiefly in Erie and northern Warren Counties. They are of Devonian age and underlie the Pocono group where it has not been removed by erosion. The Devonian rocks consist of shales and sandstones that are conglomeratic in places. The sandstones yield small to moderately large supplies of somewhat mineralized water at moderate depths, but where deeply buried, the water is likely to be highly concentrated.

Over much of western Pennsylvania the rocks appear to be nearly flat-lying although the regional dip of the beds is about 15 to 20 ft. per mile. In the southeastern part of the area, however, the rocks

have been folded into a series of northeasterly trending anticlines and synclines on the flanks of which the beds dip as much as 1,000 ft. per mile. The regional dip of the rocks in western Pennsylvania is such that a large spoon-shaped basin is formed, with the middle of the spoon at about the southwest corner of the state. On the southeast edge of the spoon the beds are sharply folded.

The structural features in the southern part of the area give rise to artesian conditions, particularly in the synclines; and many flowing wells have been drilled in these areas.

Sources of Ground Water

The source of potable ground water in western Pennsylvania is the precipitation that falls on the area. Doubtless, some of the deeper-lying rocks contain sea water that was entrapped at the time the sediments were deposited, although the composition of these deep-seated brines, in many places, differs from that of present day sea water. Ground water levels are in a constant state of fluctuation as a result of variations in recharge from rainfall and changes in rate of discharge, both natural and artificial. In this area, ground-water levels are generally high in the late winter or early spring and low in the fall of the year. The amplitude of this fluctuation in any one year may be as much as ten or more feet, and over a period of years, the fluctuation may be considerably greater.

In the unconsolidated deposits in western Pennsylvania ground water occurs in the pore spaces between the particles of clay, silt, sand, and gravel. Not uncommonly, about one third of the volume of these sediments is pore space. In the consolidated bed-rock the water fills either the pore spaces, such as in sandstones, or the joints, cracks, and bedding planes in sandstones, shales, or thin-bedded limestones. Deposits of sand and gravel produce the largest quantities of ground water in western Pennsylvania, and sandstones are the next most important source.

Well Casings

Most of the public water supplies utilizing ground water depend on wells, although springs are used by some of the smaller communities or as an auxilliary supply for some of the larger ones. The methods used in drilling and setting casings and well screens often determines whether the maximum yield is obtained or whether the well is adequately protected against possible sources of contamination.

Wells drilled into consolidated bedrock are generally cased only near the surface, unless it is desired to shut off undesirable water at greater depths. Below the casing, the hole stands open and water may enter the well along the entire wall of the hole where the rock is water-bearing. The usual procedure is to drill into the bed-rock five or more feet and to set the end of the casing at this depth. Care should be taken to produce a water-tight seal at the end of the casing so that contamination from the surface cannot enter the well along the outside of the casing. If the casing is not seated in an impervious formation it should be tightly cemented in place. Wells penetrating rocks of low permeability often do not yield the desired amount of water. The yield of such wells can sometimes be increased by setting off explosives in the well. In some instances, this procedure has materially increased the yield of the well, though in other cases, the well has been lost.

Well Screens and Drilling

One of the chief problems of well construction in many parts of western Pennsylvania is the placing of screens in wells ending in unconsolidated sand and gravel so as to obtain the maximum yield. Many of these wells consist of ordinary casing near the bottom of which holes have been drilled. The bottom of the casing is often left open to the formation. Maximum capacity of such a well is generally considerably less than the formation would yield to a properly screened well. Furthermore, a well with open-end casing is likely to yield water containing large quantities of sand or mud.

Many different types of well screens are on the market, which, if properly installed, will furnish much larger quantities of clear water than can be pumped from a well finished with perforated casing. In wells drilled by the cable-tool percussion method the screen is generally washed down or bailed down into the water-bearing formation. Many drillers prefer to drill and case through the water-bearing formation, set the screen inside the casing, and then jack the casing back so that the screen is exposed to the formation. Some screens are made in which as much as 20 per cent of the screen area is intake opening. The size of slot or screen opening is generally determined after making a mechanical analysis of the water-bearing material. A size of slot that will allow about 50 to 70 per cent of the water-bearing material to pass through the screen is commonly chosen.

Many large capacity wells have been drilled by the hydraulic rotary method in which mud-laden fluid is forced into the hole to prevent the wall of the hole from caving. The hole is drilled with a large diameter, or is later under-reamed to a large diameter at the depth where the screen is to be placed. After the screen has been set, large quantities of well selected uniform size gravel are placed outside it. This produces a mass of highly permeable material outside and thus reduces the velocity of the water as it enters the well.

Properly constructed wells are subjected to a period of development after the screen has been placed. Numerous methods, such as pumping at a high rate, surging, back-washing, or using compressed air have been used. All such methods are for the purpose of removing the fine material in the water-bearing formation outside the screen and thus producing a more permeable arrangement of the formation around the screen where the water velocity is highest. The period of development usually lasts from a day or so to a few weeks, depending on the local conditions. Adequate and proper development is essential if the maximum yield is to be obtained. Unfortunately, many well drillers and well owners have underestimated the desirability of adequate development.

On completion, a well should be thoroughly tested to determine its true capacity. During the pumping test, water level measurements should be made in the well to determine the drawdown for the particular pumping rate. The pumping test should be continued until the drawdown has reached essential equilibrium. Too short a test may indicate a greater yield than can be maintained during continuous pumping. Such tests often extend over a period of a few weeks.

From the start of drilling to the end of the pumping test, careful records should be kept of all operations. All too often difficulties are encountered which cannot be successfully remedied because of incomplete or inaccurate records of drilling, developing, or testing.

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Discussion by *H. C. Kneeland*.* Rather than present a formal discussion of Mr. Leggette's paper, the writer, here, wishes to present some additional data on the underground flow in the Allegheny and Ohio Valleys.

While strictly limited to the valleys of streams draining the glaciated areas, the water here considered has been a prolific source of water supplies for numerous communities and industrial plants. It is available in large volume and initial purity at practically any point where a well can be drilled in islands or flood plains in the lower Allegheny and Ohio Valleys and those of many of the tributaries. This water has all the characteristics of a true ground water and is so constant in quality and volume that a short study of its origin may be interesting.

The encroaching ice of the first glacial period crossed Lake Erie, dammed all of the northward flowing streams, reversed them in their flow and established a completely new drainage system. Blocking of the outlet of the Clarion-Allegheny-Ohio River, which was originally northward through the present Beaver Valley, backed up a lake from Wheeling to Oil City in the broad shallow valleys and, when this lake overflowed and cut a new outlet to the Mississippi System, a deep and narrow valley was eroded in the floor of the older ones.

The second glaciation left an extensive till sheet covering all the northern counties of New York, Pennsylvania and Ohio, filled and obliterated the valleys of small streams and delivered millions of tons of sand and rock fragments to the main stream to be carried away, rolled into gravel and deposited as the gravel beds which we now know throughout the entire length of the Allegheny and Ohio River Valleys.

The original deposits varied in depth from 80 ft. in the Pittsburgh District and 150 ft. at Cincinnati to 400 ft. in the Conewango Creek between Chautauqua Lake and Warren, Pa., but the rivers have subsequently removed some of this material and deepened their beds to leave extensive islands and flood plains later covered with a deep deposit of alluvial silt.

In the glaciated areas the valley train of sand and gravel becomes integral with the ground morain or till sheet which varies from 30 to more than 100 ft. in depth and acts as an enormous sponge to collect rain water and deliver it by percolation to the gravel deposits of the southward and westward flowing streams.

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* Water Analyst, Pittsburgh.

Having entered the gravel deposits below the river, this water becomes essentially a ground water and flows the length of the valley completely divorced from the surface river flowing above it, without any mingling of waters except where pressure from rain percolating through flood plains expels gravel water upward into the surface river. There is no reversal of flow or infiltration of river water even where hard pumped wells are located directly in the river channel.

The static level or water table of the gravel water under islands and flood plains exactly corresponds to river level with a very slight lag only at times when the rivers rise rapidly. During the 1936 flood many wells from this source were completely submerged below the river but delivered clear, uncontaminated water with unchanged analysis as soon as pumping could be resumed.

Analysis of gravel water is not affected by river stage or weather changes but has a consistent seasonal change, concentration of soluble minerals being greatest in November and December and least in April and May. Dissolved oxygen, carbon dioxide and organic nitrogen are very low in the water, with alkalinity ranging from 65 to 140 p.p.m. and hardness from 150 to 300 p.p.m. Iron content is usually below 0.2 p.p.m. but manganese sometimes reaches 4 p.p.m. and gives considerable trouble through the staining of linens. In some localities where blast furnace slag has been used for a fill, the alkalinity and hardness are markedly increased, but this water responds very well to softening by the zeolite process.

The volume of water available from the gravel deposits is ample for all reasonable demands and is inexpensive to obtain as depth of wells need not exceed 60 ft. Wells of the gravel envelope type easily yield up to 5 m.g.d. and there is practically no interference from wells placed closely together if they are aligned at right angles to the flow of the river.

Relation of Waste Disposal to Western Pennsylvania Water Supplies

By C. H. Young

PRACTICALLY all of Western Pennsylvania, with the exception of the small Lake Erie area in the extreme northwestern part, is located on the Ohio River drainage basin. The Ohio River upstream from the Pennsylvania-Ohio state line has a total drainage area of 22,117 sq.mi., of which 15,571 sq.mi. are located in Pennsylvania. Principal tributaries with their drainage areas are shown in Table 1.

The main river system—the Ohio, Monongahela and approximately the lower half of the Allegheny—is used for navigation, and is provided with locks and dams. The area is one of the important industrial districts in the United States. West Virginia and Pennsylvania jointly produce nearly 60 per cent of all of the bituminous coal mined in the country. A substantial part of this production, as well as an important part of the country's steel production, is located on the drainage area. During 1937, a good steel year, about 40 per cent of the tonnage of steel ingots and castings produced in the United States came from this section and contiguous territory in the Mahoning Valley, draining into Pennsylvania, and in the Weirton-Wheeling district, adjacent to Pennsylvania. The steel tonnage for the district, for the same year, exceeded that of Germany or of England and France combined. While many other productive activities center there, e.g., most of Pennsylvania's oil fields, coal and steel and related industries have a most important bearing on its general prosperity.

It is estimated that the total population on the drainage basin above the Pennsylvania-Ohio state line is about four million. This includes the population on the watersheds in New York State, north-

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A paper presented on September 19, 1940, at the Western Pennsylvania Section Meeting, New Castle, Pa., by C. H. Young, District Engineer, Pennsylvania Department of Health, Meadville, Pa.

ern West Virginia and the Mahoning Valley in Ohio, all of which drain into Pennsylvania. Practically 85 per cent of this total population resides in Pennsylvania.

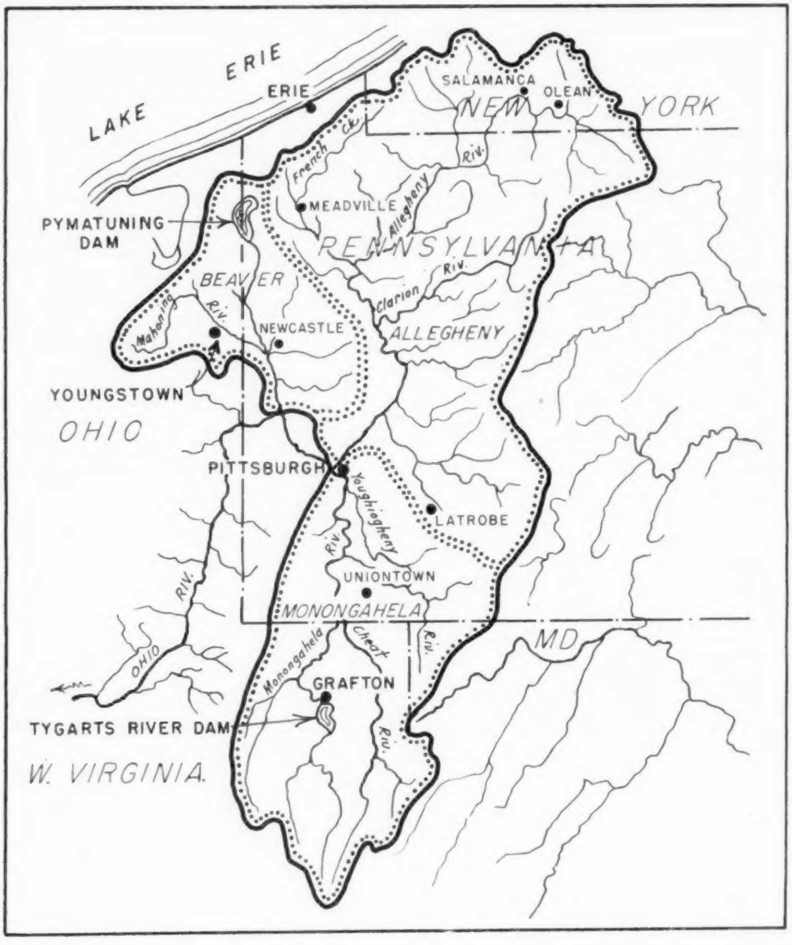

FIG. 1. Western Pennsylvania Drainage Basins

Most Pennsylvania municipalities of 2,000 or more population are provided with sewer systems, the sewage of which is discharged treated or untreated to waters of the state. Pittsburgh, with a population of approximately 675,000, located at the confluence of the Allegheny and Monongahela Rivers, is the largest municipality on the basin.

Sources of Supply

Municipal water supplies are taken from the Allegheny, Monongahela, Beaver and the Ohio River proper, in that order of importance according to population served. Only one public supply, serving some 6,000 persons, takes its raw water from the Ohio in Pennsylvania.

All water supplies taken from the rivers or their tributaries, with the exception of a limited number of upland impounded supplies which are chlorinated, are provided with water filtration plants.

In some areas, ground-water supplies are used; the principal among these supplies being those from the river gravels in and along the Ohio River below Pittsburgh, those from glacial drift in northwestern Pennsylvania and a number of small supplies from different geological formations scattered throughout several parts of the drainage basin. Many of the ground waters obtained require purification to reduce objectionable mineral constituents such as hardness, iron, manganese or combinations of these.

TABLE 1

Drainage Area of Western Pennsylvania River Systems

RIVER SYSTEMS	DRAINAGE AREAS	
	sq.mi.	
Allegheny		11,705
(1) Kiskiminitas	1,891	
(2) Clarion	1,232	
(3) French Creek	1,246	
Monongahela		7,340
(1) Youghiogheny	1,732	
(2) Cheat	1,427	
Beaver		3,140
(1) Mahoning	1,050	
(2) Shenango	1,090	

Among the polluting agents which have an effect on the production of safe and potable public water supplies are: sewage, acid mine drainage, acid pickle liquors, phenols, tar compounds and oils, wastes from tanneries and paper mills and similar establishments, and salt water from oil production. The increase of low river flows by releasing stored water from dams has a material bearing on this problem.

Sewage Disposal

Many studies have established the importance of sewage disposal to public health and to the safety of public water supplies. If the limits of sewage pollution are approached, the solution is through the treatment of upstream sewage to such a degree as to reduce the load on the downstream purification plants. Streeter (1) has suggested the following limits for bacterial loads: for conventional rapid sand filters with post-chlorination, 5,000 coliform organisms per 100 ml.; and for conventional rapid sand filters with pre- and post-chlorination, 20,000 coliform organisms per 100 ml. Frequently, the highest bacterial loads on downstream water works on the drainage area occur during the months of the higher stream flows and/or lower water temperatures. This is true for the alkaline streams, as well as for those which are acid on low flows.

The Monongahela, Youghiogheny, Kiskiminitas and many tributaries, in the southwestern part of the state, are usually acid, due to the discharge of mine drainage. These acids act as germicides so that high bacterial loads at the filtration plants are seldom encountered.

In the northwestern part of the state, the principal streams and most of their tributaries, including the Allegheny above the confluence of the Kiskiminitas, French Creek, and the Clarion and Beaver Rivers are alkaline. All but the Clarion River are used as sources of public water supplies. Because of heavy bacterial loadings on downstream water works, and/or the lack of diluting water on low flows, many sewage treatment works have been constructed. All municipalities on French Creek, for instance, treat or soon will treat their sewage before it is discharged to the stream. Likewise, all Pennsylvania municipalities on the Beaver River watershed have provided sewage treatment works. Satisfactory conditions on the Beaver, however, can only be secured by the cooperation of the State of Ohio in cleaning up the Mahoning River, since it has a definitely malign influence on the Beaver. A program to secure the early treatment of sewage discharged to the Allegheny River upstream above the Kiskiminitas is being carried out. Several municipalities have recently completed sewage treatment plants along this portion of the river, and others are either constructing intercepting sewers or working on other programs toward the desired end. Sewage treatment plants have also been constructed by many municipal-

ities on the tributaries of these streams; and, in all, very substantial progress has been made toward the solution of the problem.

Acid Mine Drainage

Acid mine drainage plays a very important part in the problem of pollution control and water supply. Large quantities of such drainage are discharged from the bituminous coal fields, principally in southwestern Pennsylvania (approximately 6,000 mines) and northern West Virginia. The effects of these discharges can be noted on the Monongahela, which is acid at its mouth most of the time; the Allegheny, below the confluence with the Kiskiminitas, which likewise is acid on some low flows; and the Youghiogheny and the Kiskiminitas which are acid streams. The acid condition of the Monongahela and Allegheny, in turn, affects the Ohio River, which, in the vicinity of Pittsburgh, is acid on low flows.

Mine drainage contains iron salts and mineral acids which act as a germicide. The acid condition of these streams, which receive heavy sewage burdens, is responsible for the low coliform indices found at most water works intakes. The East Liverpool, Ohio, water filtration plant, for instance, is about 40 miles downstream from the location where the concentrated untreated sewage from over one million persons is discharged and reports (2) an average coliform index for the raw Ohio River water for a ten-year period of only 9,420 per 100 ml.

The free mineral acids, the iron salts and the navigation pools, serving as settling and re-aeration basins, contribute to the reduction of the effect of the sewage so that a very low dilution factor, about 1 cu.ft. per sec. of stream flow per 1,000 persons discharging sewage, is found in the upper Ohio—not considering the oxygen requirements of industrial wastes which are substantial. It is surprising to note the amazing clarity of the Ohio River on low flows below Pittsburgh, and the lack of visible evidence of either a nuisance or an exceptionally heavy upstream pollution.

Stream flows remain substantially the same over the years, although population, amount of sewage discharged, acid mine drainage and industrial wastes have increased. Doubtlessly, a material reduction of acidity in the streams in this area, especially in the Ohio River, would result in a serious stream condition.

Almost unbelievable amounts of acid are discharged in mine drainage to this drainage area. Herndon and Hodge (3) estimate

that 1,750,000 lb. of concentrated acid per day are discharged to the Monongahela watershed in West Virginia alone; and about 4,000,000 lb. of acid are estimated to be discharged in the mine drainage on the drainage area in Pennsylvania. Mine sealing work has proved its effectiveness in lessening the acid load on these streams, as has been found through the reduction of acidities on low flows in both the Monongahela and lower Allegheny Rivers in the past several years.

Figure 2 shows the relationship of alkalinity, acidity and hardness of the Allegheny River water at Aspinwall, by years, from 1909

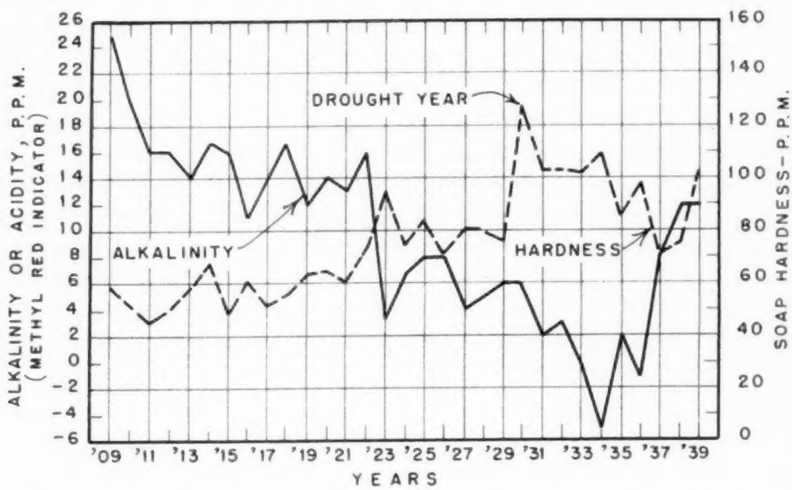

FIG. 2. Alkalinity and Hardness of the Allegheny River at Aspinwall (Pittsburgh)

to date. It will be noted that there has been a definite change in the alkalinity trend in the past five years. This can be attributed to higher low stream flows, less coal being mined, the abandonment of unprofitable or worked-out mines and to mine sealing. Mine drainage effect on the public water supply is attained through reducing the alkalinity of the stream to the point, in some cases, of producing an acid condition by increasing the hardness and the iron, manganese, aluminum and total solids contents of the river water. The uncontrolled increase in the amounts of mine drainage discharged to the streams of this area will result in the production of a degree of acid pollution where the cost of purification or the chem-

ical character of the finished water will be such that new supplies at considerable cost will have to be considered.

Acid Pickle Liquors

The tonnage of steel produced in the United States has varied greatly from year to year, with the production trend being upward. Likewise, steel production during any year may vary considerably from a very high to a very low rate, or vice versa. Acid, principally sulfuric, is used to clean the surface of the steel in an operation which is commonly referred to as pickling. After the pickling process, the spent acid, containing free acid and an appreciable quantity of ferrous sulfate, is discharged from the pickling tanks. Steel production, such as tin plate, strips and sheets for furniture, refrigerators and automobiles, wire, conduits and pipe, has increased greatly in use in relatively recent years—so much so, that a new method, called continuous pickling, has come into use for certain products. The amount of acid used varies generally from 50 to 85 lb. per ton of steel handled. A limited study shows that 10 to 21 per cent of the total acidity is discharged in the rinse waters following the pickling operation, whereas the balance of the total acidity remains as spent pickle liquor. Pickle liquors from batch pickling usually contain from 1 to $3\frac{1}{2}$ per cent free acid and 15 to 22 per cent ferrous sulfate. The free mineral acidity of pickle liquors from continuous picklers generally is two or three times that of the batch pickler, and contains lower amounts of ferrous sulfate.

The amount of acid discharged through spent pickle liquors is very much smaller than that from acid mine drainage. It is estimated that of the total acidity discharged to this river system above the Pennsylvania-Ohio State line, only about 10 per cent is from spent pickle liquors, including again the acid wastes from the Mahoning Valley and the Weirton-Wheeling district. This amount, and its effect on the streams, varies greatly depending upon business conditions. High rates of steel production and acid used have occurred at times of low river flows, with correspondingly greater effects on the streams.

Limited facilities for acid disposal or recovery have been provided; and the work of the American Iron and Steel Institute on spent acid recovery is awaited with interest.

Figure 3 shows the effects of the spent pickle acids discharged to the Beaver River and tributaries during the years 1929 through

1932. This covers a range from a good business year to a depression year. It will be noted that the alkalinity of the Beaver is greatly reduced by spent acid wastes during high rates of steel operation. The results of poor steel business in 1932 are apparent by the nearly normal river alkalinities.

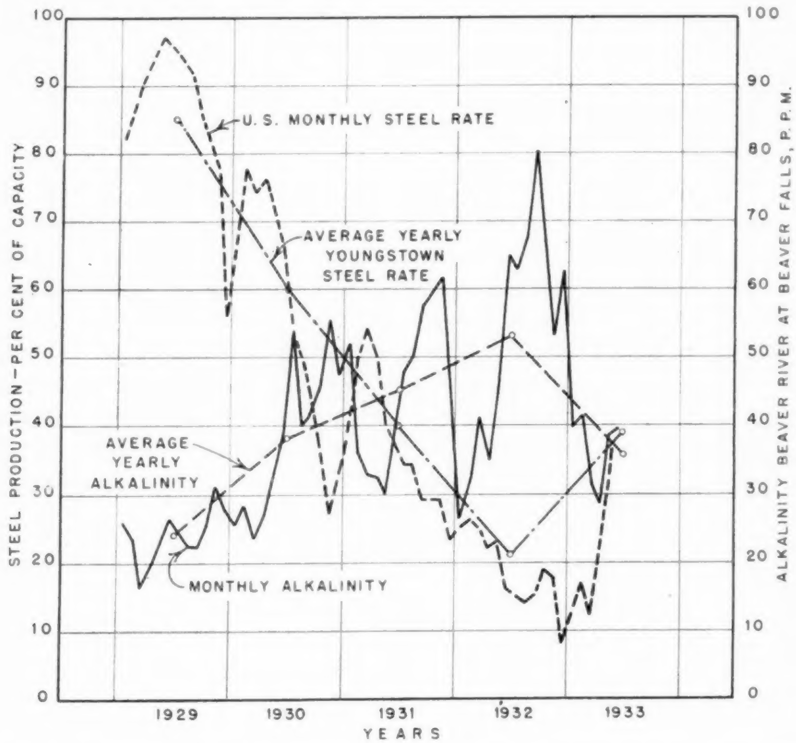

FIG. 3. Relation of Steel Operations to Alkalinity of Beaver River; average yearly Youngstown steel rate is based upon production at Youngstown, Warren and Campbell, Ohio, and Farrell and Sharon, Pa.

The effect of spent pickle liquors on the streams that serve as sources of public water supplies is similar to that of acid mine drainage, except in those cases where the irregular discharge of pickle liquors has produced marked fluctuations in alkalinity, pH, hardness, CO_2, iron and manganese at downstream water filtration plants. This latter effect is particularly objectionable, especially where the filtration plant may operate under a purification burden.

Tastes and Odors

The objectionable tastes and odors from the public water supplies of this drainage area originate principally from by-product coke or similar works, some industrial establishments, swamp lands on certain parts of the area, or from combinations of these causes. There are nine by-product coke plants, one of which was the world's largest plant even before its capacity was doubled to 30,000 tons per day. Four plants are located on streams in adjacent states but whose watersheds discharge into Pennsylvania. The largest plant is located at a point where any discharge of wastes is into a river navigation pool from which a large municipal water supply is obtained. In addition, two large water works take water from the next pool downstream. Taste and odor difficulties from this source are seldom experienced at these water works plants. In 1927, West Virginia, Ohio and Pennsylvania entered into an agreement to regulate the discharge of wastes from these plants and its effect has been reasonably successful.

The wastes from tanneries and a large paper mill on the Clarion have at times produced colored water and taste and odors at the downstream Allegheny River supplies. These conditions have been related to heavy rates of withdrawal of water from the Piney Power Dam on the Clarion River at times of low river flow in the Allegheny River. Subsequent improvements in waste disposal at the upstream industries and an understanding with the power company relative to high drawdowns of the dam during low river flows have since minimized difficulties from this source.

The discharge of large amounts of mine drainage, pickle liquors, or both, combined with the re-use of surface supplies for cooling and processing water results in imparting tastes, classified as a steel mill flavor, to a water supply derived from the stream. This may not be objectionable to the regular user, but it is invariably noticed by one unaccustomed to the supply.

Swamp water, algae and decomposition taste and odors, at certain seasons of the year, are also factors in taste and odor control at some plants in the area. Practically all water works obtaining water from surface streams are provided with taste and odor control facilities. Some plants are equipped with facilities of various types, and in the main accomplish satisfactory results.

The Beaver River water works has a most difficult taste and odor

problem. During January of this year an average of 600 lb. of activated carbon per million gallons of water treated was used. Figure 4 shows the seasonal occurrence, the relationship of the Pymatuning Dam releases and the importance of the problem of tastes and odors at the Shenango and Beaver River water plants. The graph is based on partial chemical costs by months, over a period of years, for two large water filtration plants, one near the mouth of the Shenango and the other near the mouth of the Beaver River. The Shenango River plant experiences the greatest taste and odor difficulty and has the highest purification costs during the months of higher water temperatures. Low flow control, with the

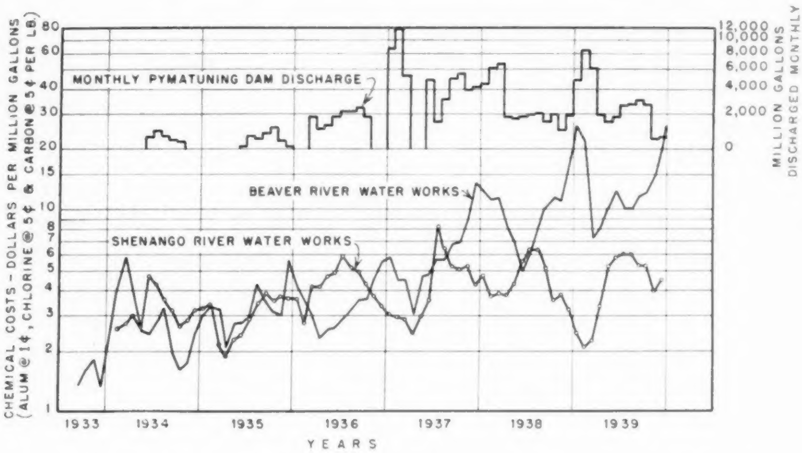

FIG. 4. Partial Chemical Costs of Shenango and Beaver River Water Purification Plants

release of water from the Pymatuning Dam, has been of value to the plant, although this has resulted in higher purification costs. The taste and odor difficulties of the Beaver River water works are somewhat similar to those of the Shenango plants in that increased summer taste and odor difficulties and higher purification costs have followed the use of water for flow control from the dam. The big problem at the Beaver River plants, and the important difference between the plants, is that the worst taste and odor conditions and the highest chemical costs occur during the winter months. This is because of the discharge or release of objectionable taste and odor materials to the Mahoning River in the Youngstown, Ohio, district.

Salt Water

The large portion of Pennsylvania's oil fields located on this drainage basin includes an estimated 80,000 producing oil wells. Salt water invariably accompanies the production of gas and oil. Over the years, the discharge of salt water has not produced the difficulties that might be expected. The largest public supply to have been adversely affected is that of Butler, Pa., where some years ago the water company obtained an order from the court restraining the discharge of salt water above its impounding dams. Improperly capped or plugged oil and gas wells that have been abandoned have affected, and are still affecting, the saline content of the ground water supplies in certain areas; and there is no practical treatment for the removal of salt from the public water supply. The effect of salt water discharged to the streams, however, is gradually being corrected through depletion of the oil reserves.

Flow Augmentation

The storage of flood waters in dams and the release of regulated amounts of water during low stream flows can serve a number of useful purposes, and is especially important to the highly industrialized districts. So far, two dams, the Pymatuning and Tygarts River Dams, have been constructed for this purpose on the Ohio River drainage basin, and others are proposed.

Pymatuning Dam was constructed by the State of Pennsylvania in 1933–34 on the head waters of the Shenango River to maintain regulated low flows at Sharon of not less than 200 cu.ft. per sec., the lowest flow of record there having been 8 cu.ft. per sec. The Tygarts River Dam was constructed by the Federal Government on a tributary of the Monongahela River above Grafton, W. Va. This is a multiple purpose dam combining flood and navigation control. A smaller dam has been constructed on the Mahoning River in Ohio, primarily for the purpose of increasing the industrial water supply. The features of interest of these dams are shown in Table 2.

Nine additional flood control dams are proposed as authorized by the 1936 or subsequent flood control acts. These dams (4) are proposed at the following locations:

(1) Allegheny River above Warren;
(2) Tionesta Creek above Tionesta, under construction;

(3) Crooked Creek near Ford City, under construction;
(4) French Creek near Cambridge Springs;
(5) Conemaugh River above Saltsburg;
(6) Loyalhanna Creek near Saltsburg, under construction;
(7) Mahoning Creek near McCrea Furnace, under construction;
(8) Red Bank Creek near Mayport; and
(9) Youghiogheny River near confluence, under construction.

It will be noted that all except the last named dam are to be constructed on the Allegheny River basin. The combined gross capacity of these reservoirs is about 2,000,000 acre-ft., of which the Allegheny Dam above Warren will have approximately one-half. About half of this capacity is to be used for low flow control. The Youghiogheny reservoir, with a capacity of 254,000 acre-ft. is also intended for use as a multiple purpose reservoir.

TABLE 2

Design Features of Pymatuning and Tygarts Dams

	CAPACITY FOR FLOW CONTROL	DRAINAGE AREA AT DAM	REGULATED DISCHARGE	APPROX. ALKALINITY OF RELEASED WATER
	mil.gal.	*sq.mi.*	*cu.ft. per sec.*	*p.p.m.*
Pymatuning	74,000*	150	200	38
Tygarts	32,500	1,200	350	6

* With 2-foot flash boards; not all capacity available for flow control.

The alkalinity of most surface streams varies inversely with the flow. Impounding flood water means storing waters of lower alkalinity and hardness. Figure 5 shows the relationship of alkalinities to stream discharge at Sharon on the Shenango River, before and after the construction of the Pymatuning Dam. With a regulated flow of 200 cu.ft. per sec. from the Pymatuning Dam and 350 cu.ft. per sec. released from the Tygarts River Dam, this is equivalent to the addition of approximately 30,000 and 10,000 lb. of alkalinity per day, respectively, over normal unregulated low flow conditions. It will be noted that the total pounds per day of alkalinity added, by flow augmentation, to this river system during low flow periods is small in comparison to the estimated acid load.

Flow augmentation on the Beaver River has contributed to the improvement of the chemical quality of the river water from the standpoint of hardness and mineral content as well as in smoothing

out fluctuations produced by the intermittent discharge of industrial wastes, especially acid wastes. On the other hand, it has intensified the summer taste and odor control problem as well as increasing chemical costs at downstream water plants. Possibly the greatest benefit of the Tygarts River Dam will be noted on the upper reaches of the Monongahela, although it will be especially important in diluting the river water on low flows and thereby lowering the hardness, iron, manganese and aluminum content, tastes and odors, as well as acidity. The normal low flow of the Monongahela is about 300 cu.ft. per sec. It will be seen that the added dilution by flow augmentation will double this figure and will, there-

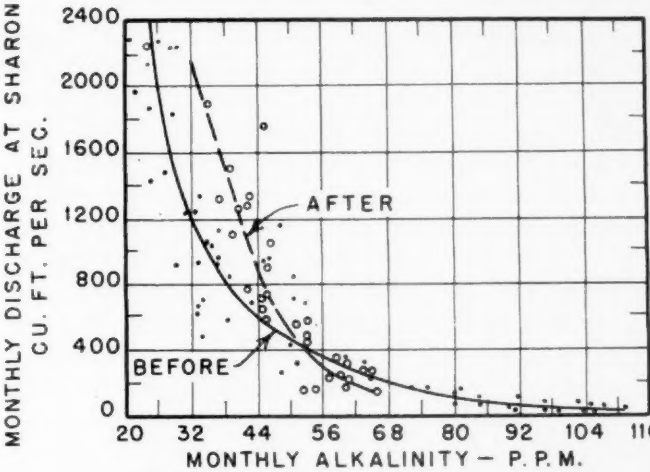

FIG. 5. Alkalinities and Stream Flows of Shenango River before and after construction of Pymatuning Dam

fore, be a valuable water works help. Bouson (5) reported on the 1930 drought effect on raw water quality at the South Pittsburgh water plant where a peak hardness of 550 and a total acidity of 70 p.p.m. was experienced. The 1930 low flow was 170 cu.ft. per sec. and the dilution value, by increasing the river flow three times with a soft water, under these conditions, is apparent.

The 1930 drought-year low flow of the Allegheny River at Pittsburgh was 650 cu.ft. per sec. It is expected that the proposed Warren Dam will increase this flow to 2,065 cu.ft. per sec. Studies made by the Army engineers indicate that if approximately half of the Warren Dam capacity is used for flow control, there will be

sufficient alkalinity released to neutralize the acidity of the lower Allegheny during a year of as great acidity as that which occurred in its peak year, 1934. Since 1934, there has been a reduction in acidities of the lower Allegheny during low flows; consequently, the added dilution from the proposed Warren Dam or from other of the flood control dams, now under construction or proposed, will be a substantial further aid. In all probability, flow augmentation will mean the extension of sewage treatment to this part of the river.

The Allegheny River minimum runoff per square mile of drainage area is over twice that of the Monongahela River. The Allegheny drainage area is about 50 per cent larger than the Monongahela and has less coal mining development. The flood control program proposes the construction of eight dams, (four are now under construction) on the Allegheny basin. These reservoirs will have most of the capacity of the ten-reservoir system and with several exceptions will receive water of higher alkalinity than the Tygarts Dam. Possibly several of the Allegheny basin reservoirs can be used in addition to the proposed Warren Dam for low flow augmentation. Studies should be undertaken to determine the added flow augmentation value and the extent of corrective measures necessary on the drainage areas to permit additional reservoirs to be used for flow control. These combined considerations indicate that the Allegheny continues to hold future water supply possibilities.

Water Works

There are many interesting water purification plants, varying from the simple untreated well or ground water supply to the large complicated plant treating a water difficult to purify. A plant may handle a water varying in extreme conditions from a low acidity and a high hardness, iron, aluminum or manganese content to a fairly soft alkaline water. In some instances the acid salts of iron and aluminum present in the raw water are used almost exclusively as the coagulants.

There has been a definite trend over the years toward the installation of water softening plants, both for the surface as well as the ground water supplies. Western Pennsylvania has a large number of water softening plants. Generally, lime-soda softening for the surface supplies and zeolite softening for the ground-water supplies are installed. Some of the most recent plants use a com-

bination of lime and zeolite. All filtered surface waters are chlorinated; and most plants handling a raw water of high bacterial load practice both pre- and post-chlorination. Winter-time pre-chlorination has not been possible in such cases at all times due to the presence of intermittent objectionable taste and odor materials in the raw water. Where winter-time pre-chlorination is needed as an added purification factor, the presence of taste and odor materials is most objectionable if they require the discontinuance of pre-chlorination to prevent taste and odor intensification.

Activated carbon is widely used with success for taste and odor control. The Beaver River water plants have, during winter months, used one of the highest sustained carbon doses on record in the country. Ammonia, aeration and break-point chlorination are all used, some of these very widely. It should be noted that none of the taste and odor procedures in use has proved completely effective for the more severe taste and odor conditions. This emphasizes the need either to improve the taste and odor control procedures at the water purification plant or to reduce further the discharge of objectionable wastes to streams. The winter-time tastes and odors from the Mahoning in Ohio are the major source of trouble, in this respect, in western Pennsylvania.

Reducing the effects of mine-drainage and spent-acid wastes means lower purification costs for the water works, as well as a lessening of operation problems. Low flow augmentation likewise means improved raw waters with less fluctuations and lower purification costs unless taste and odor problems are intensified. This latter consideration may be important where waters containing algae are released into streams acid on low flows.

Conclusions

This brief review should indicate that there is a problem in water supply and waste disposal in this area which is different in many respects from that in any other locality. Probably there is no other area of equal size in the world that produces as great a tonnage of materials and goods as is produced here. Its economic existence is built largely on heavy industry, including the natural resources; and the development of many years has left its effect on certain of the streams of the area. Over the years, the population and the quantity of sewage, industrial wastes and mine drainage, discharged to the streams, has increased. The value of the work accomplished,

to date, through sewage treatment, industrial waste control, mine sealing and low flow control is apparent through the reversal of the down trends of the indexes of pollution over a number of years. There is reason to believe that the major streams in this area have been and can yet be still further improved as sources of water supply of good quality. Certainly it is possible to prevent progressively less satisfactory conditions in these streams. A general unified program which will include the completion of sewage and industrial-waste treatment improvements where required, continuation of mine sealing, and the construction and use of low flow control reservoirs are required to accomplish the purpose.

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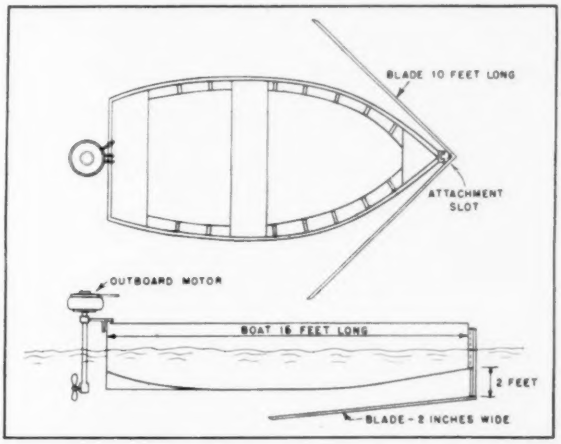
Underwater Cutting Tool for Aquatic Plants

By B. B. Harris and J. K. G. Silvey

A SUCCESSFUL method of removing the emergent portions of aquatic plants from impounding reservoirs, by the use of a cutting blade attached to the prow of a flat-bottomed boat, has been developed in the Southwest. The equipped boat, shown in the accompanying figure, is propelled by a 24-horsepower motor at about 15 m.p.h.; and, depending upon the density of the vegetation,



a path varying from 50 to 150 yd. long can be cut before the propeller becomes entangled with the floating parts of the plants to the extent that it must be stopped and cleaned. Though only the aboveground portions of the plant may be removed in this way, it is expected that, by frequent cutting, the underground portions will die from lack of proper nutrition.

A contribution by B. B. Harris, Dean of the College and Director of the Department of Biology, and J. K. G. Silvey, Associate Professor of Biology, North Texas State Teachers College, describing equipment developed for use at Denton, Tex.

Photo-Electric Control for a Drinking Fountain

By John E. Kleinhenz

TO GIVE customers a thrill by the most up-to-date method of getting drinking water, the Indianapolis Water Company installed in its office lobby the drinking fountain with photo-electric control illustrated herewith. By way of contrast an antique wooden pump was also placed in the lobby.

Photo by courtesy of *Indianapolis Star*, September 9, 1940

The special equipment used for this installation was supplied by the Electronic Control Corp., 2667 East Grand Boulevard, Detroit, Mich. The light relay is #LR 237; the light source is #LS 227; the solenoid operated valve is of a type made by a number of manufacturers. The diagram on the following page shows how the equipment was arranged.

Readers wishing further information may write John E. Kleinhenz, Publicity & Advertising, Indianapolis Water Company, Indianapolis, Ind.

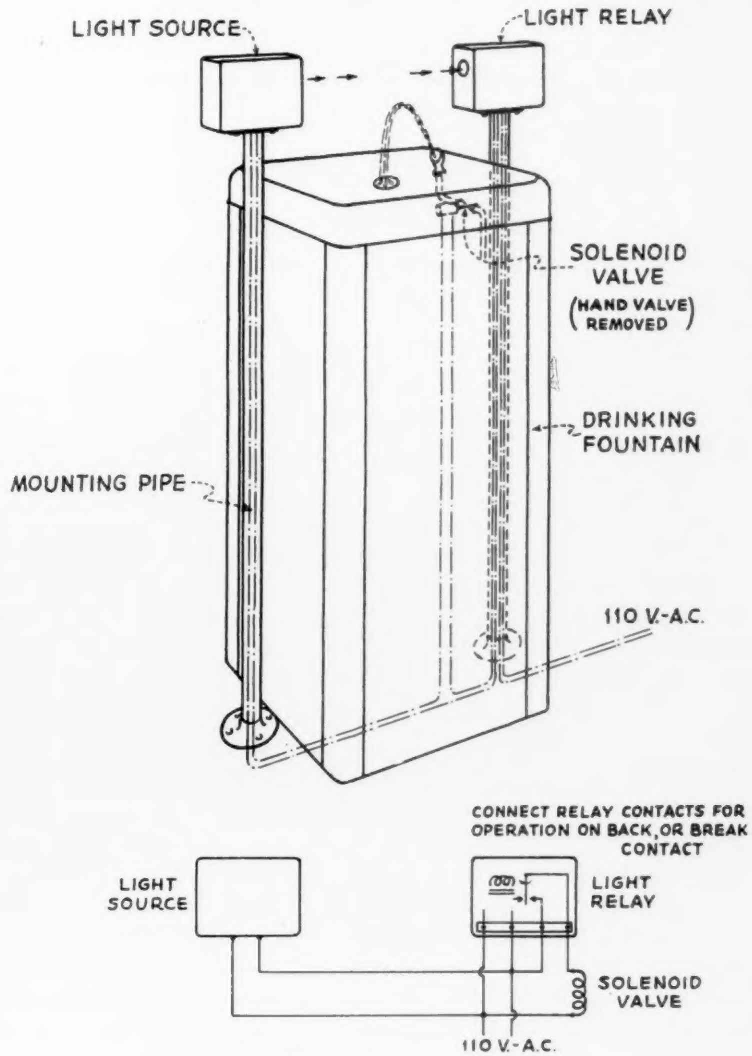


PHOTO-ELECTRIC CONTROL FOR DRINKING FOUNTAIN

Chemical Feeders

By L. E. Harper

THE first dry chemical feeders were placed on the market in the early 1920's. At that time orifice boxes for feeding solutions were the rule of the day. The heavy cast-iron Pittsburgh Filter Co. dry chemical feeder was one of the first and one of the best available, although its accuracy was limited somewhat by its water-motor drive. Wallace and Tiernan feeders with a revolving disk, utilizing the Booth patent, were available, but they were dusty to operate. Savage-Gauntt made a screw- or auger-type feeder in which the screw was ratcheted around at the desired speed by means of an adjustable Pitman rod. International Filter Co. and the Omega Machine Co. came into the dry feeder field in about 1925. Considerable progress in design and appearance was made until about 1930, when most companies put housings around the feeding mechanisms to confine the dust and to improve the appearance of the machines. The vibrator feeders made their appearance in about 1932 or 1933.

Although minor refinements have been made, there have been no fundamental improvements in accuracy or performance of volumetric chemical feeders since about 1930, nor is there likely in the future to be much improvement other than in appearance, for where greater accuracy, dependability or automatic control are desired, it is necessary to turn to gravimetric feeders. Probably not over a dozen installations of gravimetric feeders were to be found in the United States prior to 1935 when the Omega Machine Co. began promoting their sale, but then development came quickly, International, Jeffrey and Syntron following with designs of the same character.

A frank appraisal of the history of chemical feeding equipment for use in water purification and sewage treatment plants to date must

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A paper presented on April 25, 1940, at the Kansas City Convention by L. E. Harper, Omega Machine Co., Kansas City, Mo.

convince any unbiased person that the industry is just now emerging from the "Dark Ages" as far as chemical feeding equipment is concerned. The writer regards about 75 per cent of the chemical feeders being used today as obsolete, inadequate, time-consuming and wasteful. By this, it should not be inferred that gravimetric feeders are the only satisfactory type. Volumetric feeders definitely have their place, and in some instances should be used in preference to all other types.

One very important development in the use of volumetric feeders that should be mentioned is the use of scales in conjunction with them. Since 1928, volumetric feeders on scales have been available

FIG. 1. Installation of Large Size Volumetric Feeders with Hopper and Dial Scales

from any of several manufacturers at a nominal extra cost. Scales on which chemical feeders are mounted are used to calibrate the feeder by means of a short test ranging from 1 to 10 min. to determine the proper feeder setting, and to check the accuracy or delivery of the feeder by checking the amount fed over periods of an hour or longer. The keeping of a daily record of the total amount fed is a check against the amount purchased.

The involved procedure necessary to calibrate a volumetric feeder for a single setting is its greatest disadvantage. First, a testing pan must be located for catching a sample of the material. The pan, usually, is not furnished by the manufacturer of the feeder,

but is a makeshift device made up at the plant when the need for it arises. Then, scales must be secured. Nine times out of ten, a trip must be made to the laboratory to get the laboratory scales, since testing scales not specified in the order are not furnished with the feeder.

Reference must be made to a calibration chart, and the feeder set at what is hoped will be the correct rate of feed. If the testing pan will hold only about a pound, and the rate is 60 lb. per hr., a one minute test is the longest that can be run. On a short test run like this, an error of 2 seconds would amount to over 3 per cent, or an error in weighing of 1 oz. would be equivalent to an error of 6 per

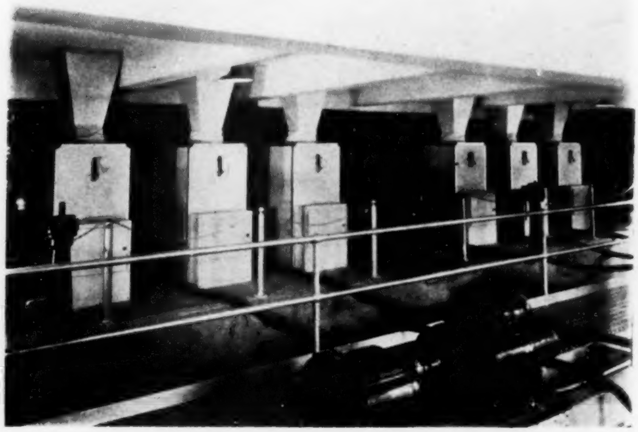

FIG. 2. Vibratory Feeder Enclosed in Cabinet for Dust Elimination

cent. Laboratory scales, too, are most frequently calibrated in grams, so that it would be necessary to convert from grams back to ounces or pounds to obtain the desired calibration in pounds per hour or grains per gallon.

If the first setting is not correct, another setting must be made and the process repeated. This is an inaccurate, time consuming and wasteful method.

A feeder mounted on a pair of ordinary platform scales holds considerable advantage over the ordinary volumetric type. The operator merely has to fill the hopper, balance the scales and start the feeder, noting the time on a slip of paper. He may then go

about doing any other duties which might take his attention for a few minutes. When he returns he must only note the time, snap off the feeder motor, and again balance the scales, noting on a separate scale beam, if desired, the exact amount of material delivered.

Inadequacy of Simple Volumetric Feeders

The difficulties which may arise with a simple volumetric feeder are indicative of its inadequacy. Assume, for instance, that an operator started up his plant and obtained a good floc with a certain feeder setting. When the plant was operating smoothly he would

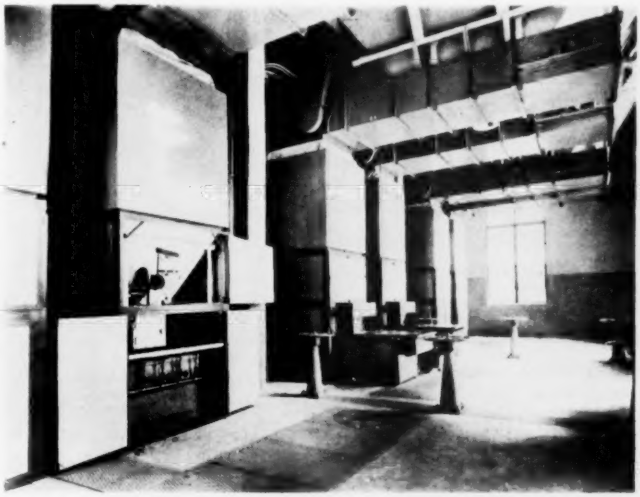

FIG. 3. View of Large Batch Type Gravimetric Feeders; showing 2 slakers and large mixing chamber under the feeder with its doors open (Cincinnati, Ohio)

go about his other duties. Returning later to check on the coagulation basin, he might find that the floc had disappeared. A check of the discharge line from the lime feeder and alum feeder could show that they were feeding correctly. An examination by means of a sight glass or float could show that the desired number of gallons were delivered in the period of operation, and no further determination of the trouble would be possible.

Though the feeder would be operating satisfactorily at the time, without scales no determination of the actual number of pounds of lime fed would be possible.

An accident such as the temporary obstruction of the feeding orifice, for instance, could not be discovered in any way; and on the basis that a laboratory check indicated a deficiency of lime, the operator would naturally increase the rate of feed to prevent reoccurrence of what he has diagnosed to be the trouble. Such a procedure would, of course, waste lime unnecessarily.

Advantages of Gravimetric Feeders

If the lime feeder in the plant had been a gravimetric feeder, it would have been possible for the operator to set the feeder by means of a hand wheel or a dial to exactly the desired feed in pounds per hour, without any preliminary calculations of calibrations. Thereafter, the feeder would weigh out the lime at the desired rate in pounds per hour and record the weight of lime fed.

Accuracy is, of course, important, and that is one of the qualities of gravimetric feeders that is usually mentioned first. There are, however, other advantages equally as important:

1. In case of stoppage of the feeder, or if the feeder hopper has been allowed to run empty, an alarm sounds immediately to notify the operator that something is wrong.

2. Material totalizers which will keep a record of the number of pounds that have passed through the feeder can be incorporated in the apparatus.

3. The feeders can immediately be pre-set to deliver the required amount of material in pounds per hour. They will maintain this feed, regardless of changes in density of the material being handled. No calibration is required.

4. They can be equipped with chart type mechanical operation recorders which keep a permanent record of the performance of the feeders and the amount of material fed.

5. Accuracy will depend to some extent, upon the skill of the operator, but usual guarantees are that they will deliver within 2 per cent plus or minus the rate of feed for which they are set; or that they will maintain a uniform rate of feed within 2 per cent plus or minus. One per cent may be obtained by careful operation.

In comparing volumetric feeders on scales with gravimetric feeders, it may be noted that, when used with volumetric feeders, the scales make possible only post mortem examination and indicate to the operator what adjustment should have been made if he had desired the correct treatment. In gravimetric feeders, the scales control the rate of feed at all times by automatically adjusting it.

Solution Feeders

Any kind of chemical used in water treatment, as a solution or as a suspension, can be fed by means of a suitable feeder. Where very small feeds are required, the liquid feed is usually the desired method. Certain chemicals such as ferric chloride, sodium hexametaphosphate and sodium silicates must be fed in the solution form.

There are four principal types of solution feeders:

1. *Decanter type feeders*, such as those manufactured by Proportioneers, Inc., American Water Softener Co. and Omega Machine Co., are probably the most accurate. Certainly they have the

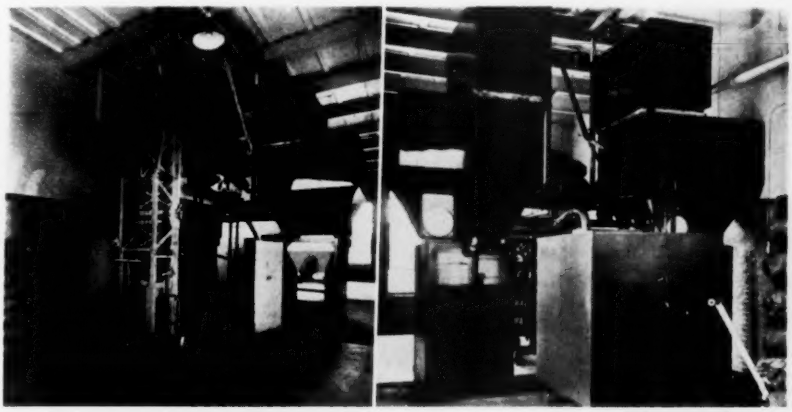

FIG. 4. Two Views of Gravimetric Feeder Installation; showing canopies and empty carton hampers which with dust-removing apparatus make plant dust-proof (Cincinnati, Ohio)

least amount of mechanism in contact with corrosive materials. An alarm which will ring when the tank runs empty, and a gage to indicate the amount of material remaining in the tank at all times may be provided with these feeders. A totalizer which will record the total number of gallons that have passed through the feeder may also be installed very easily.

2. *Diaphragm pump units* such as those built by Proportioneers and International Filter Company may be used for feeding corrosive solutions, where the point of application is above the supply tank. These pumps may take the suction from a tank of any size and the tank can be refilled while the pumps are in operation.

3. *Archimedes wheel type feeders* such as those made by Wallace

and Tiernan and Omega Machine Company are gravity type feeders which must be placed above the point of delivery.

4. *Orifice type feeders* require no motors for their operation. They are, however, to be distrusted on very low feeds due to the ease of stoppage of small orifices and the large error introduced by small accretions on the orifice.

Need for Standard Specifications

One reason for the use of so much obsolete chemical feeding equipment today is the use of loosely drawn specifications. The success or failure of any given installation of chemical feeding equip-

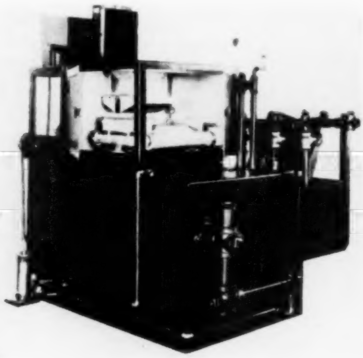

Fig. 5

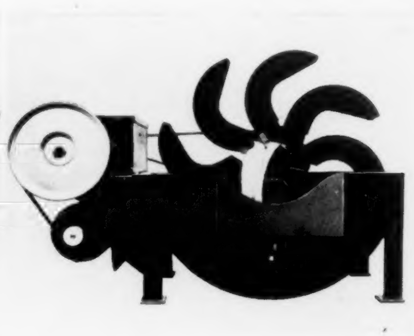

Fig. 6

Fig. 5. Belt Type Gravimetric Feeder with special stainless steel insulated mixing chamber and automatic water control to mixing chamber

Fig. 6. Archimedes-Wheel Type Feeder in Operation; showing dipper buckets which hold fixed amount of material regardless of depth of solution in tank

ment is more dependent upon the auxiliary equipment furnished with the chemical feeders and the arrangement of its installation than it is on the use of any particular make of feeder. Many manufacturers have urged that the specifications be copied verbatim from their catalogs. This may tend to limit competition somewhat, but it certainly is not conducive to a good installation, for seldom are two installations exactly alike.

Some example of items to be noted in writing chemical feeder specifications which may insure the success or failure of the installation, regardless of the type or manufacture of machines used, are:

1. *A hopper capacity* of at least an eight hour's supply at normal

rate of feed and the number of cubic feet the hopper should hold should be specified. As a matter of fact, it is best practice to have the hopper hold eight hour's supply at the maximum expected rate of feed. Extension hoppers for chemical feeders are inexpensive, and it costs no more to store a portion of the chemical in the feeder hopper than on the chemical storage room floor.

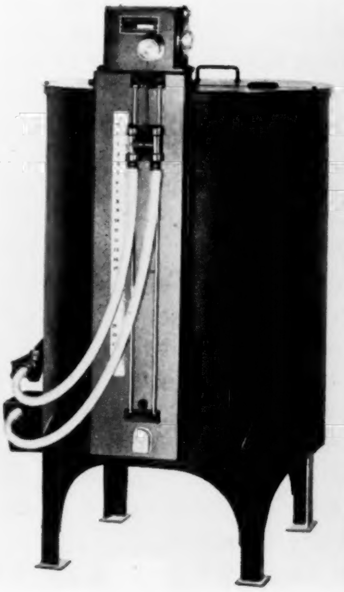

FIG. 7

FIG. 8

FIG. 7. Standard Precision Feeder with scale to show amount fed and empty-hopper alarm buzzer

FIG. 8. Belt Type Gravimetric Feeder mounted below soda-ash storage bin; showing glass doors which permit easy inspection and supervision (Lansing, Mich.)

If the plant is a small one that is operated only for a maximum period of ten or twelve hours a day, the chemical feeder hopper should hold a full day's supply. It is not wise to specify a chemical feeder with a feeding range of from 1 to 100 lb. per hour and a hopper capacity of 3 cu.ft. This means that at the maximum rate of feed, if the feeder is feeding lime, the hopper must be refilled about every 70 minutes. The plant operator usually has enough to do without

filling the hopper of the chemical feeder every hour or so and then cleaning up the dust and mess occasioned thereby.

2. *The feeding range* in a chemical feeder that is actually going to be required should be specified. Almost all manufacturers' catalogs give the minimum and maximum rates for which any size feeder may be used. These figures should not be taken as recommendations for the minimum and maximum feeding rates for any one feeder. A feeding range of from 1 to 100 lb. per hour is wider than that encountered in any usual plant. Normally a feeding range of 40 to 1 is sufficient. If it is possible that at a later date a wider feeding range, that is a higher capacity, will be required, the specifications should state that the feeder be capable of an adjustment to deliver at the rate of 25 to 50 per cent higher at some date.

3. *Mixing chambers* of adequate capacity should be provided with chemical feeders so that the chemical will be dissolved if it is capable of being dissolved, or thoroughly soaked if it is a material such as hydrated lime which does not dissolve readily. An appreciable amount of chemical is lost if this provision is not made. A suitable specification would be that the material be retained for at least 3 min. in the mixing chamber of the feeder at the maximum rate of feed, when using sufficient water to give a 10 per cent solution. A 5-minute retention would be even better. It should be kept in mind, however, that with 3 min. retention at the maximum rate of feed, the retention in the mixing chamber at average rates of feed will probably be from 6 to 10 min., and this is desirable.

4. *Adequate dust removal equipment* should be provided. Plant operators are entitled to humane treatment and should not be forced to sniff the dust laden air that billows up when a sack of lime, alum or carbon or soda ash is dumped into a chemical feeder hopper. The operator can wear a mask, but that will not prevent the lime dust from settling on him when he perspires in the summer time, and the respirator will not help him keep dust out of the machinery nor help preserve a clean appearance of the plant. Adequate dust removal equipment that will confine and filter out the dust is available at a nominal cost and should be specified. As a matter of fact, in industrial plants, either the use of dust removal equipment or the wearing of respirators is mandatory by state health inspection departments.

5. *Bin gates* should be installed between chemical feeders and large storage bins holding a carload or more. They should not be used

with extension hoppers holding up to 2,000 lb. of material unless they are required by the design of the feeder, which may not be built to withstand the superimposed load of chemical.

6. *The kind of current available* for operating the feeder motors and, if desired, the type of motor, such as totally enclosed, explosion proof, 3-phase or single-phase, voltage, etc., should be specified and request should be made that the chemical feeder manufacturers state the horsepower required to operate their feeders. The feeder manufacturer should be in a position to know how much power is required to operate his machine as the fact that a large or small motor is required may have considerable weight in the selection of the feeder he will use.

Conclusions

The following conclusions concerning the specification of chemical feeders may be drawn from this survey:

1. Gravimetric chemical feeders are the preferred type of feeder to use in any water or sewage treatment plant.

2. Volumetric feeders, when used, should be mounted on scales.

3. Accessory equipment such as scales, extension hoppers, bin gates, dust removal equipment and arrangement of equipment must be covered closely by specifications, because these items more than make of equipment determine a successful installation.

A Small Pressure Filter Plant

By William W. Watkins

A SMALL municipality has just as much right to expect a generous quantity of good, pure water as any larger community. Toward this end, Richmondville, N. Y., has installed an unusually compact and economical pressure filter plant; and it is the purpose of this paper to describe that plant.

Richmondville is a typical rural village drawing its trade from a dairy and agricultural territory. In the population of less than 700 there are no large water consumers other than a milk plant and a central school.

In 1894, a reservoir with a nominal capacity of four million gallons was constructed about three miles west of the village. No attempt was made to purify the water in any way. In 1912, a so-called settling basin and small slow sand filter were constructed on the side of the reservoir. The water flowed through the settling basin and slow sand filter into the earthen reservoir and then to the village. At times of any appreciable rainfall, all surface wash was carried directly into the reservoir and from there to the village. In 1931, a dry feed chlorinator was installed on the line near the reservoir.

From time to time the system was increased until now there are approximately 8 mi. of cast-iron water main varying in size from 4 to 10 in. in diameter, 31 fire hydrants, 207 water connections and a few valves scattered throughout the village. This was the system in 1938, when the building of a new state road caused large quantities of clay to be washed directly into the reservoir and gave a turbid water in the village most of the time.

In the summer of 1938, the writer was employed by the village to design a filter plant and lay out certain other improvements for the water system. A study was made of three possibilities: driven

A paper presented on June 7, 1940, at the New York Section Meeting, Ithaca, N. Y., by William W. Watkins, Consulting Engineer, Oneonta, N. Y.

wells, pressure filters in the village, and a gravity filter plant located near the reservoir. After due consideration, the idea of driven wells was abandoned, as, among the taxpayers, there was much sentiment against wells. The question of a gravity or pressure filter was discussed at some length and a gravity plant was finally decided on, against the recommendation of the engineer. Preliminary estimates showed that the cost of the pressure plant would be slightly less than the gravity plant, but the village board felt that, although the gravity plant would have to be located at a rather inaccessible place, it had advantages which out-weighed both the cost and location.

This project and a sewer project were submitted to the P. W. A. for approval, but before any village vote had been taken. The P. W. A. then approved only the sewer project; the taxpayers, only the filtration plant; and the impasse held up all work until August, 1939. In the interim a bond issue for the entire project was voted in a village election, and a new village board reconsidered the question of plant type, finally approving the plans for a pressure filter.

Details of Project

The final plans called for increasing the raw water storage to five million gallons and constructing a pressure filter plant consisting of a master meter, chemical feed equipment, settling tanks, and four pressure filters. Two 15,000-gallon storage tanks were planned to float on the line in the southern end of the village.

Raw water storage was increased by excavating to straighten the shore line of the existing reservoir and by some minor repairs to the structure. Stream flow was directed to the newly excavated portion which, separated from the old reservoir by an earth embankment, acted as a settling basin for the heavy suspended matter. To eliminate the necessity for frequent cleaning, a drain to draw off this settled matter was provided at a low point in the basin. Additional storage capacity was also provided by the removal of several thousand yards of muck from the old basin when it went completely dry during the project.

The hardest problem facing the designing engineer was the determination of the consumption and from that of a proper design figure. There were no master meters nor any meters in the village. A rough check of the drawdown in the settling basin indicated a consumption of 200 gallons per minute. This did not seem reason-

able with a population of only 626 and with no large industrial users. Pitot tube checks showed a consumption of 100 to 150 g.p.m. with the creamery running. Opening a fire hydrant wide increased the flow to slightly over 200 g.p.m. Assurance was given that there were no sizable leaks in the system; but, before the investigation was completed, one large leak was found and others have since come to light. With storage planned within the village and a chlorinated bypass in the filter plant, it was decided that 200 g.p.m. would be sufficient for the design of the filters and settling tank. At no time since the meter was installed has the consumption exceeded a peak load of 125 g.p.m., the daily average being approximately 75 g.p.m.

The board was fortunate in being able to obtain a site near the village limits directly over its water main. After the building was constructed, a cut was made and a 6-inch Sparling compound meter set in the line, with a "T" connecting the raw water to the settling tanks and a straight-away valve for bypass. The proportional feed hypochlorite machine was hooked to the meter and the hypochlorite solution was so introduced into the main that if it should ever be necessary to open the bypass or if there should be any leakage, all of the water going to the village would be chlorinated automatically, thus eliminating any of the ordinary bypass hazards.

Chemical Feeders

Since one of the major objections to pressure filter plants is their inability to compensate for sudden changes in flow, due to direct connections on the plant, it was felt necessary to obtain some form of chemical feeding equipment which would be absolutely automatic, changing with the flow on the system. After a careful study of the different types of available equipment, it was decided to use the Wallace & Tiernan water-driven hypochlorite feeders. These feeders are controlled by a shaft hooked to the compound meter in such a manner that both the large and small meters drive the one shaft to the chemical feeders. Four feeders were supplied and any one of them can be used for any of the chemicals. It is possible to use one machine for pre-chlorination and one for post-chlorination, while the other two are being used for the coagulant and alkali treatments. The four feeders were mounted on one base and, as stated above, driven by a common drive shaft. They have worked very satisfactorily since their installation, with the exception of the

machine feeding soda ash. To feed enough soda ash, it was necessary to have a very concentrated solution. The machine and hose "froze" and gave trouble. This condition was entirely eliminated, however, by placing two cams on the shaft, thus pumping the liquid twice as rapidly and allowing a reduction in the strength of the solution. All chemical solutions are mixed in 50-gallon stone crocks on an elevated platform. They are then drawn from the mixing crocks into the crock from which the machine feeds.

Several connections were made at different points in the plant so that it would be possible, by the use of one of the chemical feeders, to feed chlorine or soda ash at any desired point. Experiments on the soda ash treatment before sedimentation and before filtration, and, at the present, on post-filter feeding have been conducted.

Settling Tanks

Another undesirable feature of the older pressure filter plants was a lack of any sedimentation or coagulation tanks. With maximum rate of flow based on the data obtained, a combination mixing chamber and settling tank was developed on a basis of 200 g.p.m. Two tanks, 10 ft. in diameter and 23 ft. 6 in. long were provided. The first tank was equipped with a mechanical agitator with arms and baffles placed on the inlet end. The agitator was driven by a 3-h.p., 1,200-r.p.m. motor. The speed of the agitator was reduced to $7\frac{1}{2}$ r.p.m. by a suitable reduction gear. Water flows out the back end of the first tank into the back end of the second tank, and from there through it to the filters. The tanks can also be by-passed for cleaning or repairs. The tanks and agitator were designed in conjunction with The Permutit Company of New York, which furnished them.

The question of detention in the settling tanks was given considerable study before a definite decision was reached as to the length of time necessary for proper treatment of the water. The detention period in the mixing chamber and settling tanks was also planned on a basis of 200 g.p.m. At this rate, a detention of 17 min. in the mixing compartment and 2 hr. in the settling tanks obtained. This was felt to be sufficient, as any additional overload in the village would draw from the storage tanks. The present rate of flow is from 75 to 100 g.p.m. The detention period by saline checks shows more than 4 hr. at 100 g.p.m. flow. As a result of this long detention, the water is practically free from floc.

Pressure Filter Battery

The filter battery consists of four vertical steel pressure units (Fig. 1). Each unit has a diameter of 72 in. and the shells have a 5-foot straight side. The filters were designed to operate at a rate of 2-g.p.m. per sq.ft. or 57 g.p.m. per filter. The filter shells were designed for a working pressure of 75 lb. and in other respects, to conform with accepted practices. All of the valves were standard approved rising stem type.

The filters were equipped with automatic air relief valves, duplex loss of head pressure gages, sampling cocks and butterfly control

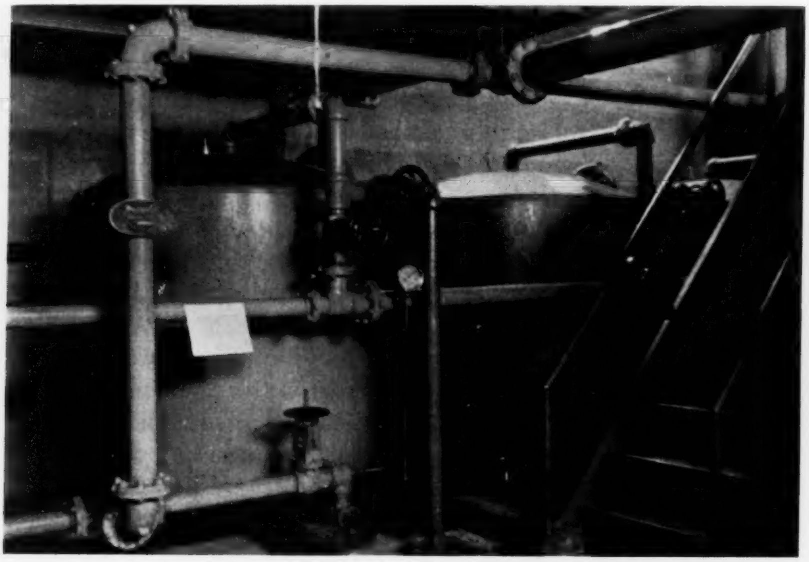

Fig. 1. Part of the Battery of Pressure Units

valves for regulating backwash rate of flow. In front of each filter unit, a concrete box, with a partition containing a 6-inch weir, was constructed. The backwash water discharges into one side of this box and the butterfly valve is set to give a pre-determined head on the weir. If the water rises above that head, the rate of wash is cut down and thus a uniform rate is obtained. The drain connected to the sewer flows from the other side of the weir.

The chief difference between these filters and the standard pressure filters is in the under-drain system. Each filter is provided with a patented filter bottom consisting of an all cast-iron deflector-dis-

tributor filtered water collecting system. The plate is cast into the filter when it is constructed and the gravel is supported by the plate, wash water rising around the outside and flowing upward through the gravel.

The filter media for the plant consist of 24 in. of fine sand with an effective size of 0.4 to 0.5 mm. and a uniformity coefficient of 1.5; 6 in. of coarse sand with an effective size of 1.0 mm. and a uniformity coefficient of 1.5; 4 in. of $\frac{1}{8}$- to $\frac{1}{4}$-inch gravel; 4 in. of $\frac{1}{4}$- to $\frac{1}{2}$-inch gravel; 8 in. of $\frac{1}{2}$- to 1-inch gravel; and 4 in. of 1- to $1\frac{1}{2}$-inch gravel, all of which makes a total depth of 50 in. of aggregate.

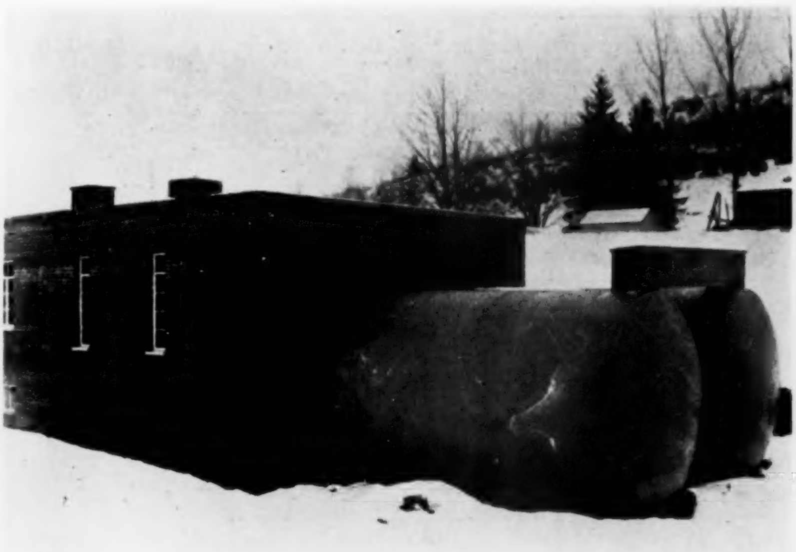

FIG. 2. Rear View of Plant, Showing Pressure Settling Tanks

To keep down the cost of the entire project and to get a compact filter plant, the settling tanks were located partly outside the building (Fig. 2), with just enough space for connections inside. A building 35 ft. 5 in. by 23 ft. 4 in., with two elevations was designed. The first or basement floor carries the chemical feeding equipment, the filters, the foundation and part of the settling tanks. The oil-heating plant was also located in one corner of this floor. Only a portion of the second elevation was floored giving an opportunity to see, from the upper level, the operation of the chemical feed equipment. Storage space for chemicals was provided on this floor and

TABLE 1

Cost Data on Pressure and Gravity Type Filter Plants in Richmondville, N. Y.

ITEM	GRAVITY PLANT ESTIMATED	PRESSURE PLANT	
		Estimated	Actual
Settling Tanks	$5,400.00	$5,610.00	$5,696.00
Filters Installed	5,000.00	3,100.00	3,378.00
Meter & Chemical Feeders	1,800.00	2,300.00	2,300.00
Piping	600.00	900.00	957.63
Pump	500.00		
Painting Pipe, etc.	424.00	300.00	300.00
Building	5,000.00	3,900.00	3,900.00
Extras	936.00	805.00	100.33
	$19,656.00	$16,915.00	$16,631.96
Reservoir, Storage Tank, Engineering, etc.			10,075.13
Total Cost of Project			$26,707.09

TABLE 2

Operating Data on Pressure Filters at Richmondville, N. Y.

	JANUARY (10 DAYS)	FEBRUARY (29 DAYS)	MARCH (31 DAYS)	APRIL (20 DAYS)	AVERAGE
Average amount of water treated daily, *gal.*	90,715	94,145	96,838	96,335	94,508
Wash Water, %	1.23	1.34	0.94	1.02	1.13
Alum, *g.p.g.*	0.78	1	0.8	0.85	0.86
Soda Ash, *g.p.g.*	0.82	1	1.19	1.93	1.23
Chlorine Used, *p.p.m.*	0.60	0.56	0.47	0.45	0.52
Res. Chlorine, *p.p.m.*	.18+	.17+	.15+	.15+	.165+

TABLE 3

Data of Chemical, Physical and Bacteriological Tests at Richmondville, N. Y.

TEST	FEBRUARY			MARCH			APRIL		
	Raw	Applied	Filtered	Raw	Applied	Filtered	Raw	Applied	Filtered
Turbidity, *p.p.m.*	12.00	4.00	0.33	17.00	4.10	0.25	100.00	10.00	0.00
Color, *p.p.m.*	19.00	7.00	0.33	21.25	5.75	0.25	40.00	10.00	0.00
Odor	V-2	V-1	0	V-3	V-2	0	V-2	V-1	0
Alkalinity, *p.p.m.*	49.0	41.0	53.0	37.5	29.0	44.5	16.0	10.0	34.0
CO_2, *p.p.m.*	4.00	9.00	4.00	4.00	11.00	2.75	4.00	10.00	0.00
pH	7.3	6.9	7.4	7.2	6.6	7.5	6.9	6.3	8.8
Total Hardness, *p.p.m.*	49.0	53.0	54.0	64.5	59.0	63.5	32.0	32.0	32.0
Bacteria per ml.	13.0	4.0	2.0	2.4	4.0	1.0	83.0	15.0	1.0
Coliform Organisms	+	−	−	+	−	−	+	−	−

the elevation was so located, with reference to the ground elevation, that material could be unloaded from trucks with a minimum of effort. Located in one corner of the second elevation is a combined office and laboratory. The laboratory has been equipped with sufficient chemicals and apparatus to do simple routine work.

Cost Data

Table 1 gives a comparison of the cost of a small pressure plant and that of a similar size gravity plant. The estimated cost of the gravity plant is that used when the project was set up for P. W. A. The figures were considered extremely low by the P. W. A. authorities, although they did agree to accept them. The estimated cost of the pressure plant was within $300 of the actual cost of the plant. Assuming, then, that the estimate for the gravity plant would be within $300 of actual cost, the saving effected in building the pressure plant would amount to about $3,000.

The data shown in Table 2 give some idea of how the plant is operating. The average quantity of water treated is much below anything anticipated. Since the figures were tabulated, however, there has been a considerable increase in the quantity of water being consumed. It is now running well above 125,000 gal. per day. The unusually low percentage of wash water is believed to be due to the long detention period in the settling tanks. The amount of chemicals used is about what is to be expected with water of the type being treated. Only once, on March 31, 1940, was there any really bad water to be treated and at that time, the effluent was of as good quality as with normal raw water. The alum dosage during the period jumped from less than a grain to slightly over two grains and then gradually went back to normal.

Chemical and bacterial results as shown in Table 3 are the composite results of daily chemical checks at the plant and weekly chemical and bacteriological checks in the author's laboratory.

The main point of this paper is that it is possible for a small community to have a reasonably priced filter plant by making use of the compactness and economy of a pressure filter. It is economical to operate, as only one part-time man is required to obtain efficient operation; and it can be safely operated by a man with only a modicum of experience.

The author wishes to acknowledge the courtesies of the New York State Health Department, Wallace & Tiernan, Inc., The Permutit Co., and the village officials, especially Robert Grieve, who supplied a great deal of preliminary and operating data.

Effect of Ammonia on the Germicidal Efficiency of Chlorine in Neutral Solutions

By George R. Weber, Richard Bender and Max Levine

REPORTS in the literature on the relative germicidal efficiencies of chlorine and chloramine are conflicting; and the use of available chlorine as an index of probable germicidal efficiency has recently been questioned by a number of investigators.

Charlton (1, 2) and Rudolph (3), working with a spore forming organism, *Bacillus metiens* (hereinafter *B. metiens*), observed killing times of 20 sec. to 70 min. when employing calcium hypochlorite in a concentration of 1,000 p.p.m. available chlorine, the killing time being an inverse function of the hydrogen ion concentration.

Gerstein (4) in 1931, studying Lake Michigan water, reported that the concentration of ammonia markedly influenced the germicidal efficiency of chlorine. Thus, on the addition of 0.4 mg. chlorine to 0.1 mg. ammonia per liter, the residual chlorine after 30 min. was 0.34 mg. per liter and 0.26 per cent of the bacteria survived. Increasing the NH_3 content to 0.3 mg. per liter and using the same chlorine dosage did not affect the residual available chlorine, which was then 0.35 mg. per liter; but the rate of killing "*B. coli*" was markedly reduced as indicated by the survival of 6.8 per cent of the organisms. In the absence of ammonia, a residual of but 0.19 mg. chlorine per liter after 30 min. reduced the number of survivors to 0.003 per cent.

In contrast to these data, Rideal (5) found that, employing the Rideal-Walker test, "*B. typhosus*" was killed in 10 min., but not in 5 min., using electrolytic chlorine in a concentration of 1 part in 20,000, whereas in the presence of one equivalent of ammonia the same result was obtained with 1 part of available chlorine in 70,000.

A contribution of the Iowa Engineering Experiment Station and the Industrial Science Research Institute, Iowa State College, Ames, Iowa. The study was supported in part by a fellowship established by the Wallace & Tiernan Co., Inc.

Race (6) found that upon the addition of 0.2 p.p.m. NH_3 to 0.4 p.p.m. available chlorine as bleach a reduction of 99.3 per cent of "*B. coli*" was effected in one hour as compared with 67.7 per cent when using chlorine alone. Residuals were not reported.

It was felt that a series of experiments, properly controlling such factors as reaction (pH), temperature, concentration, oxidizable substances in solution, etc., would throw some light on this controversy. *B. metiens* spores were particularly well adapted to this work for several reasons: (1) higher concentrations of reactants could

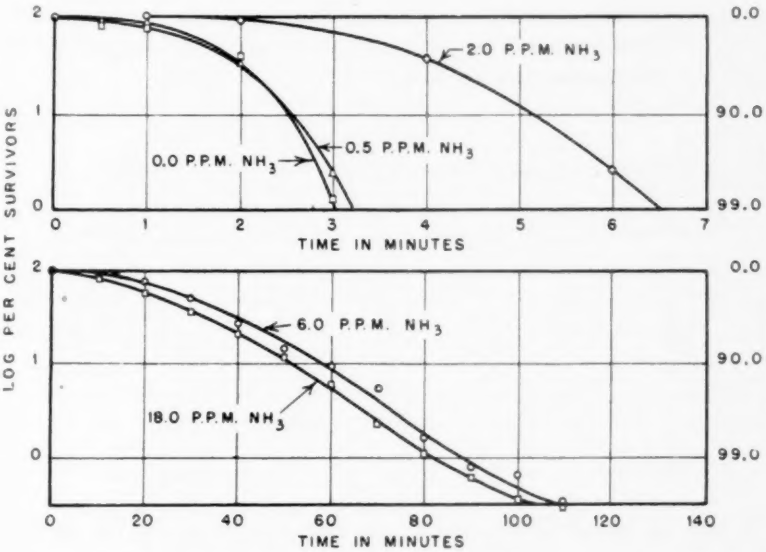

FIG. 1. Effect of Ammonia on the Germicidal Efficiency of Chlorine; available chlorine, 25 p.p.m., pH, 7.0, temperature, 20°C.

be employed; (2) the longer time required for bacterial death results in survivor curves which are more amenable to detailed study; and (3) a suspension could be prepared in which the number of viable cells and their resistance did not change appreciably over long periods of time. The following report is restricted to the results obtained at pH 7.0 and 20°C., using varying ratios of ammonia to chlorine.

A quantity of ammonia-free sterile distilled water, buffered at pH 7.0 with phosphates, was placed in a previously sterilized three-neck round bottom flask which was immersed in a water bath at 20°C. The desired quantities of ammonia, as $(NH_4)_2SO_4$, were added

and this was followed by the addition of chlorine gas in ammonia-free distilled H_2O to give 25 p.p.m. available chlorine. After a contact period of 15 min., *B. metiens* spores, to give approximately 1,000,000 per 5 ml., were introduced; and from time to time, 5-ml. portions were removed to sterile dilution water containing sufficient thiosulfate to destroy the chlorine. The numbers of surviving bacteria were ascertained by plating on nutrient agar and incubating for 24 hr. at 30°C. The time required to effect a reduction of 99 per cent was employed as a measure of germicidal efficiency and is referred to as the killing time. The survivor curves for such a series of experiments are shown in Fig. 1. The ammonia concentration, residual chlorine at the termination of the experiment, and killing times are summarized in Table 1.

TABLE 1

*Effect of Ammonia on the Germicidal Efficiency of Chlorine**

AVAILABLE CHLORINE		AMMONIA	KILLING TIME
Initial	Residual		
p.p.m.	*p.p.m.*	*p.p.m.*	*min.*
25.0	22.0	0.0	3.0
25.0	19.0	0.5	3.3
25.0	8.7	2.0	6.5
25.0	21.1	6.0	89.0
25.0	23.8	18.0	84.0

* pH, 7.0; temperature, 20°C.

It will be noted that in the absence of ammonia, the killing time was 3 min. and the residual chlorine 22.0 p.p.m.; in the presence of 0.5 p.p.m. ammonia, the residual dropped to 19.0 p.p.m., but the killing time was not appreciably altered, being 3.3 minutes. Increasing the ammonia to 2 p.p.m. resulted in a marked drop in the residual chlorine—to 8.7 p.p.m.—and an increase in the killing time to 6.5 min. When the ammonia content was increased to 6 p.p.m., which is approximately the ratio of chlorine to ammonia required to produce mono-chloramine, the residual chlorine rose to 21.1 p.p.m., while the killing time, in spite of the marked increase in residual chlorine, rose to 89 min. With the addition of still more ammonia, 18 p.p.m., the residual chlorine (23.8 p.p.m.) closely approximated the initial chlorine concentration and the killing time of 84 min. was not greatly different from that obtained with 6 p.p.m. ammonia.

The striking features are that: (1) chloramine was much less efficient as a germicide than chlorine as shown by a killing time of only 3 min. for chlorine alone as compared with 89 min. for chloramine of approximately the same residual chlorine; and (2) residual chlorine is not a dependable index of germicidal power. On addition of 2 p.p.m. ammonia, the killing time was only 6.5 min. and the residual chlorine 8.7 p.p.m., whereas with residuals up to almost 24 p.p.m., which were obtained when more ammonia was used, the killing times were 84 to 89 min. The practice, formerly recommended for water treatment, of chlorinating to a stipulated residual, as determined by ortho-tolidine, is now recognized as being open to question, for it is

TABLE 2

*Effect of Increasing Chlorine Dosage on Residual Chlorine**

AMMONIA	AVAILABLE CHLORINE	
	Initial	Residual
p.p.m.	*p.p.m.*	*p.p.m.*
2.0	5.0	4.4
2.0	10.0	8.5
2.0	12.0	8.1
2.0	13.0	5.7
2.0	15.0	1.0
2.0	17.0	2.6
2.0	20.0	5.3
2.0	25.0	10.2
2.0	30.0	15.6
2.0	40.0	24.2

* pH, 7.0; temperature, 20°C.; residual after 30 min.

pertinent to ascertain whether this residual is due to chlorine or chloramine.

In Table 2 are shown the chlorine residuals* after 30 min. contact of various quantities of chlorine with 2 p.p.m. ammonia in buffered water at pH 7.0 and 20°C. It may be seen that the residual chlorine increased until a maximum was reached and that further additions of chlorine resulted in decreases in residual chlorine to a minimum

* The available chlorine was determined by adding 5 ml. of 7.5 per cent potassium iodide (KI) solution to 100 ml. of test solution which was then acidified with 1 ml. of concentrated hydrochloric acid (HCl) and titrated with N/100 sodium thiosulfate ($Na_2S_2O_3$) employing 1 ml. of 0.25 per cent starch as an indicator.

which approached zero. With still further additions of chlorine the residuals increased in direct proportion to the chlorine added. Thus, with 5 p.p.m. available chlorine, the residual was 4.4; with 10 p.p.m.; it rose to 8.5; but on addition of 13 p.p.m. chlorine, the residual chlorine fell to 5.7 p.p.m.; and increasing the available chlorine to 15 p.p.m. resulted in a drop in residual chlorine to but 1 p.p.m. With larger initial concentrations of chlorine, marked increases in residual chlorine were obtained.

The results presented in Table 2 are shown graphically in Figs. 2 and 3. Figure 3 shows a typical break-point chlorination curve obtained by plotting the residual chlorine, after 30 min., against the chlorine added. It will be noticed that beyond the point of mini-

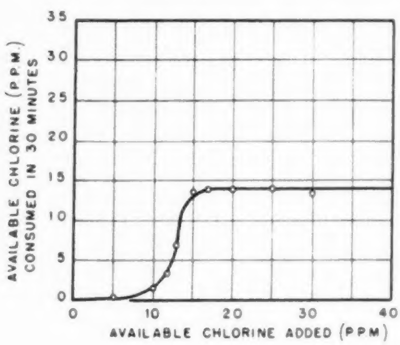

FIG. 2

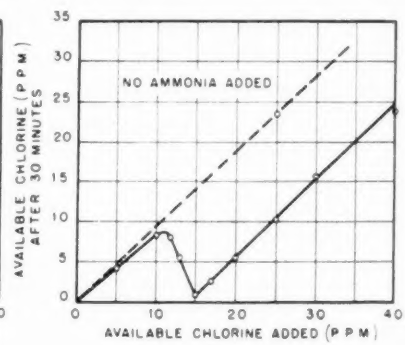

FIG. 3

FIG. 2. Effect of Increasing Chlorine Dosage on Chlorine Consumed; pH, 7.0, temperature, 20°C., NH_3, 2 p.p.m.

FIG. 3. Effect of Increasing Chlorine Dosage on Residual Chlorine; pH, 7.0, temperature, 20°C., NH_3, 2 p.p.m.

mum residual, which was obtained upon the addition of 15 p.p.m. chlorine, the residual chlorine is a straight line function of the chlorine added. In Fig. 2 is shown the relationship between the chlorine consumed in 30 min. and the chlorine added. This curve brings out the fact that when 15 p.p.m. or more chlorine is added to water containing 2 p.p.m. NH_3, (that is, beyond the break-point), about 14 p.p.m. available chlorine are used up, indicating that, in the presence of excess chlorine, the maximum amount of chlorine consumed is about seven times the ammonia present. This confirms a report by Norman (7) that, in the presence of a large excess of chlorine, ammonia is oxidized, each milligram of ammonia reacting with 6.75 mg. of chlorine. The experiments herein indicate, however,

that a large excess of chlorine is not necessary to bring about this oxidation, which went practically to completion in 30 min. when the ratio of chlorine to ammonia reached seven.

Referring to Fig. 3, it will be noticed that a residual of 5 p.p.m., for example, can be obtained by the addition of approximately 6, 13, or 19 p.p.m. chlorine. A knowledge of the germicidal efficiency of a stipulated residual resulting from the addition of different amounts of chlorine would be of considerable value and essential to an adequate interpretation of the significance of chlorine residuals. In order to obtain residuals which would approximate those in experiments previously reported in this paper, it was found necessary to

TABLE 3

*Effect of Increasing Chlorine Dosage on Residual Chlorine**

AMMONIA	AVAILABLE CHLORINE	
	Initial	Residual
p.p.m.	*p.p.m.*	*p.p.m.*
10.0	25.0	23.3
10.0	50.0	38.6
10.0	60.0	27.8
10.0	62.5	21.6
10.0	65.0	14.4
10.0	75.0	7.1
10.0	85.0	16.2
10.0	100.0	29.0
10.0	125.0	50.6
10.0	150.0	73.0
10.0	200.0	119.6

* pH, 7.0; temperature, 20°C.; residual after 30 min.

employ five times the concentrations of reactants (ammonia and chlorine) used in the experiment reported in Table 2.

As a first step in providing a series of solutions which would yield residuals of about 25 p.p.m. available chlorine, the results shown in Table 3 were obtained by determining the residual available chlorine 30 min. after the addition of various amounts of chlorine to distilled water adequately buffered at pH 7.0 and containing 10 p.p.m. ammonia. A plot of these data (Fig. 4) shows that a 25-p.p.m. residual might be expected upon the addition of: (1) 28 p.p.m., (2) 61.5 p.p.m., and (3) 95.5 p.p.m. chlorine.

To three flasks, containing 10 p.p.m. ammonia in buffered phos-

phate water, were added 28, 61.5 and 95.5 p.p.m. chlorine respectively. After a contact period of 30 min. (the residuals were 24.9, 24.2, 22.5 p.p.m. respectively), the desired number of *B. metiens* spores were introduced; and from time to time, 5-ml. portions were removed to sterile dilution water containing sodium thiosulfate. Following this procedure, the numbers of surviving bacteria were determined by plating on nutrient agar. The results of this experiment are shown in Table 4, the data of which are superimposed on the curve of Fig. 4.

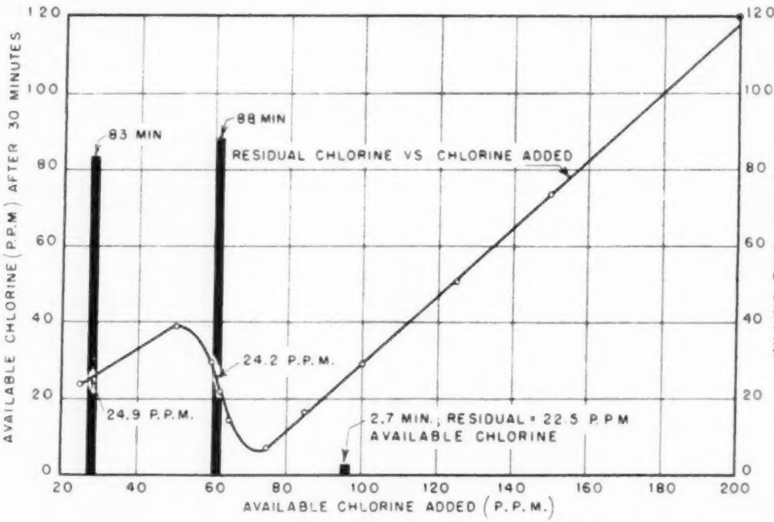

FIG. 4. Effect of Increasing Chlorine Dosage on Residual Chlorine and Germicidal Efficiency; pH, 7.0, temperature 20°C., NH_3, 10 p.p.m.

It will be noted that, when the quantity of chlorine added to produce a residual of 25 p.p.m. was greater than seven times the concentration of ammonia present (approximately 70 p.p.m.), such a residual was tremendously more effective as a germicide than was the case when smaller quantities of chlorine were added to produce similar residuals. Thus, the addition of 28.0 p.p.m. chlorine gave a residual of 24.9 p.p.m. and required 83 min. to kill 99 per cent of the exposed spores. When 61.5 p.p.m. chlorine were added to the water, the residual was 24.2 p.p.m. and the killing time was approximately the same—88 min. Upon the addition of 95.5 p.p.m. chlorine, however, the residual fell to 22.5 p.p.m. and the killing time dropped to 2.7 min.

Referring to Table 1, when chloramine was used as the germicide (25 p.p.m. available chlorine added to 6 p.p.m. ammonia), the residual chlorine was 21.1 p.p.m. and the killing time 89 min. The results in Table 4 show that upon the addition of 28.0 and 61.5 p.p.m. chlorine to 10 p.p.m. ammonia, with residuals of approximately 24.5 p.p.m., the killing times were 83 and 88 min., respectively. Since these killing times are very close to the killing time observed for chloramine in the experiment referred to in Table 1, it is inferred, on this basis, that the chlorine residuals of 24.2 and 24.9, shown in Fig. 4 and Table 4, represented chlorine in the form of chloramine.

Similarly, in Table 1, a residual of 22.0 p.p.m. chlorine, in the absence of ammonia, effected a killing time of 3 min., whereas, in Table 4, it will be noted that the addition of 95.5 p.p.m. chlorine to 10 p.p.m. ammonia produced a residual of 22.5 p.p.m., but the killing time was 2.7 min. Apparently, in the latter instance, on the basis of the killing time, the chlorine was in the form of hypochlorous acid, as of course was also the case in the experiment reported in Table 1.

TABLE 4

*Effect of Increasing Chlorine Dosage on the Germicidal Efficiency**

AMMONIA	AVAILABLE CHLORINE		KILLING TIME
	Added	Residual	
p.p.m.	*p p.m.*	*p.p.m.*	*min.*
10.0	28.0	24.9	83.0
10.0	61.5	24.2	88.0
10.0	95.5	22.5	2.7

* pH, 7.0; temperature, 20°C.; NH_3, 10 p.p.m.

A drop in chlorine residual, where an adequate ratio of chlorine to oxidizable substances has been exceeded, has recently been referred to in the water works field as "break-point" chlorination. When this point is reached, the residual drops to a minimum and the concentration of oxidizable substances is reduced; or they may be completely eliminated, with the result that objectionable tastes and odors are also markedly decreased. A slight excess of chlorine beyond the break-point, since it is due to hypochlorous acid, would be distinctly more effective as a germicide than an equal residual obtained before the chlorination break-point is reached, the latter being due to chloramine.

Conclusions

It was observed that the time to kill 99 per cent of exposed spores of *B. metiens* increased with increasing amounts of ammonia, but was neither a direct function of the available chlorine added nor of the residual chlorine.

Residual chlorine, in the presence of ammonia, was found not to be a direct function of the chlorine added, but rose to a maximum then dropped to a minimum of practically zero when the ratio of chlorine to ammonia was approximately 7 to 1, while further additions of chlorine resulted in corresponding increases in chlorine residuals.

With water buffered at pH 7.0 containing 10 p.p.m. ammonia, it was found that approximately the same residuals (22.5, 24.2, and 24.9 p.p.m. available chlorine) were obtained on addition of 95.5, 61.5, and 28.0 p.p.m. chlorine, the respective killing times being 2.7, 88, and 83 min. In view of these results it seems probable that at a point below the minimum residual, the available chlorine exists in the form of chloramines; that at the point of minimum residual probably all of the ammonia has been oxidized; and that beyond this point the residual is due to HOCl, which is particularly effective as a germicide.

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Effects of Temperature on Rate of Floc Formation

By Thomas R. Camp, Darrell A. Root and B. V. Bhoota

IT IS common knowledge among operators of water treatment plants of the rapid filter type that the coagulated water going to the filters is usually less well prepared for the filters in winter than in summer. The settled effluent from the basins contains more and finer floc and more color in winter; and more coagulant is usually required to produce a satisfactory filter effluent. These difficulties are usually ascribed to the low temperature of the water in winter, and more specifically to the effect of the temperature on coagulation.

The coagulation (or flocculation) process *per se* is the mechanism by which small colloidal or suspended particles coalesce and form larger particles. To be of value in water or sewage treatment, this process must continue until the particles are large enough to settle and filter effectively. The function of the mixing chamber or flocculator is to form the floc, but flocculation nevertheless continues slowly after the water reaches the settling basin. The primary function of the settling basin, which in many plants is still referred to as the coagulation basin, is not coagulation but settling. The complete process of conditioning water for filters is twofold therefore, consisting of both coagulation and settling.

The effect of temperature on the settling process is well known. For particles settling within the range of validity of Stokes' law (and this includes nearly all particles, except grit, dealt with in primary settling), the velocity of settling varies inversely as the viscosity of the water. Hence, other things being the same, a longer settling period is required in winter, when the water viscosity is high, than in summer, to produce the same quality of settled water. For example, at 35°F. the settling period must be 73 per cent longer than

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at 70°F. to produce the same percentage removal by settling. Since a longer settling period is not available in a plant with fixed basin capacity, the removal by settling will be less in winter than in summer. This defect may be partly or wholly compensated for by increasing the coagulant dose.

The effect of temperature upon the process of coagulation, considered separately from sedimentation, has been little studied. Two noteworthy studies which have been made are those of Leipold and Velz.

As a result of jar studies on alum floc formation at the Winnetka, Ill., filter plant, Leipold (1) reports that "there is no preventive or retarding effect on alum floc formation with low raw water temperatures." Leipold's experiments were all made with the same Lake Michigan raw water which had a turbidity of 20 p.p.m. and a pH of 7.9. An alum dose of 1.0 g.p.g. was used, but no pH adjustment was reported. The criterion by which flocculation was judged was the appearance of the floc during the 30 minutes' settling which followed a 30-minute mixing period.

In contrast with the results of Leipold's work and with the general experience of operators, Velz (2, 3) reports that higher temperatures had a detrimental effect on the removal of color from a water in northeastern New Jersey. Jar tests indicated that more alum was required at high temperatures to produce a basin effluent color of 20 p.p.m. than at low temperatures. The pH of the raw water ranged from 6.9 to 7.8 and the color from 28 to 52 p.p.m. No pH adjustment was made in these tests. It is significant that Velz reports an optimum pH in the vicinity of 5.4 for this water at summer temperatures, whereas excellent results were obtained at pH 6.7 at winter temperatures. Although no pH adjustment was made, Velz states that: "By an adjustment of the pH value, with acid, to the optimum point, the detrimental effect of high temperature can be practically eliminated."

A number of investigations have been made of the effect on coagulation with alum and iron salts of such factors as pH value, concentration of coagulant, concentration of cations and anions, velocity of stirring and shape of paddles and tank. The works of Theriault and Clark (4), Miller (5), Peterson and Bartow (6), Black, Rice and Bartow (7), Bartow, Black and Sansbury (8), Leipold (1), Nolte and Kramer (9), and Rudolfs (10) are notable in this connection. It has been shown that for coagulation of a particular water with a

particular chemical there is an optimum pH value or range within which formation of floc is most rapid. It has been shown that the optimum pH range may be extended by increasing the coagulant dose. It has also been shown that the optimum pH range may be extended on the acid side by increased concentration of polyvalent anions and on the alkaline side by increase in concentration of polyvalent cations. It has further been shown that rapidity of floc formation is increased with increased coagulant dose, and with increased speed of stirring, provided the stirring speed is not high enough to disrupt the floc particles. In most of the above studies the effects of temperature were neglected.

It is the purpose of this paper to describe experimental studies (11, 12), made by the authors, of the effects of temperature upon the the speed of formation of iron floc, and to discuss the results of these experiments.

Experimental Technique

The experimental work was performed in the Sanitary Engineering Laboratory of the Massachusetts Institute of Technology with a laboratory stirring device provided with a constant temperature water bath to accommodate six $3\frac{1}{2}$-liter battery jars (Fig. 1). The battery jars were approximately 6 in. in diameter and 8 in. high and were provided with paddles 1 in. wide by 3 in. long. The paddles were suspended within the jars to within 2 in. of the bottom, and were slightly pitched to provide an upward motion in the middle of each jar. To obviate the effect of change in speed of stirring, the paddles were revolved at a peripheral speed of 0.5 ft. per sec. in all the runs; and to avoid the effect of differences of volume of suspension, the amount of water placed in each jar was such as to produce 3 liters when all the chemicals were added.

It was shown by Theriault and Clark (4) in 1923, and it has been well established by many other investigators, that the time of formation of floc increases rapidly with increased variation of pH on either side of the optimum pH. In order to avoid the effects of pH, therefore, it was considered necessary in these studies to make every run at the optimum value. Furthermore, in order to obviate the effects of unknown or variable concentrations of flocculating ions and to avoid the presence of nuclei due to initial turbidity or color of the raw water, it was decided to use distilled water in all tests.

The distilled water was found to have an initial pH value of 6.0

to 6.2 due to absorption of carbon dioxide from the air. Because of the absence of buffer in the solution a great deal of difficulty was had at the start in the adjustment and control of the pH value when the chemicals were added. It was found by trial that this difficulty could be avoided by placing the distilled water in the battery jars and exposing it to the atmosphere for about 24 hr. before use, while it was being brought to temperature. This accomplished the absorp-

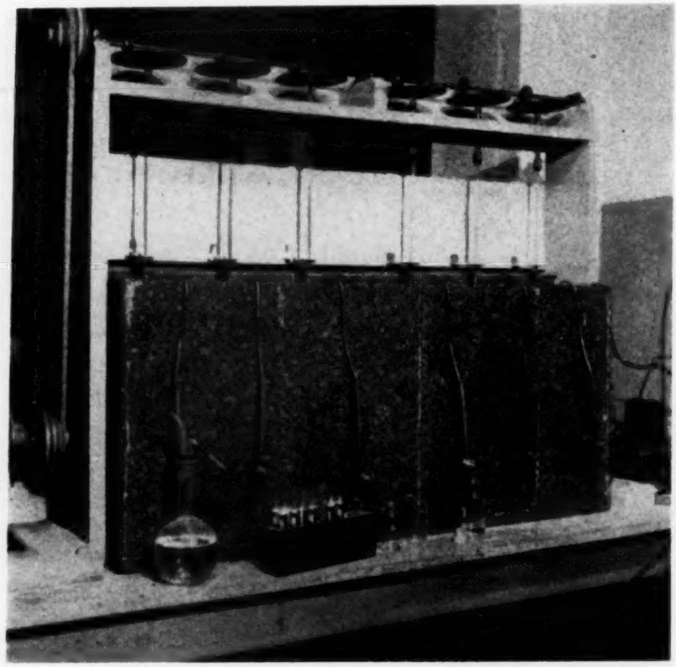

Fig. 1. Laboratory Stirring Device; showing Constant Temperature Water Bath and Sampling Tubes

tion of additional carbon dioxide, which, upon reaction with the water, yielded bicarbonate ions in the solution. The raw water used, therefore, may be defined as distilled water slightly buffered with bicarbonate to a pH value generally less than 6.0.

Since the principal measurement to be made in these experiments was the time required to form a floc, it was necessary to have a satisfactory "end-point." The "first appearance of floc" and the "first appearance of good floc" have been used by previous investigators.

Theriault and Clark (4) and Miller (5) evidently observed the floc formation by the light in the laboratory. Peterson and Bartow (6), as well as Bartow and others (7, 8), used a strong Tyndall beam passing through the jar, the jar being exposed to the light of the laboratory. Since the first appearance of floc must necessarily be a function of various factors such as concentration of light, the speed of the particles through the light and the observer's judgment of the size of the floc, it is not a good end-point for a time measurement. The first appearance, however, is quite satisfactory for determining in which of several jars operating simultaneously side by side the formation of floc is most rapid. Its use is, therefore, justified in runs for determination of optimum pH value. In these experiments optimum pH runs were made with horizontal light beams from G.E. Mazda No. 222, 2.2-volt flashlight lamps immersed in the water bath.

Inasmuch as floc is formed in a treatment plant for the purpose of clarifying the water, the speed and efficiency with which the floc settles is one of the best measures of the quality of the floc. A settling curve, showing removal during quiescent settling, plotted against time of settling is a definite characteristic of a particular floc. From the plant operator's viewpoint, two floc suspensions with the same settling characteristics may be considered as nearly identical. One of the most satisfactory end-points for measurement of time of floc formation, therefore, is the settling curve of the suspension after the floc is formed. This end-point has been used throughout these experiments. Since the settling curve for a particular floc was obviously unknown until after the floc was formed, it was necessary to reverse the usual procedure by fixing the time of formation and then measuring the magnitude of the end-point.

In order to determine the settling curve, stirring was stopped at the end of predetermined mixing periods and at those times floc was allowed to settle in the battery jars at the temperature at which coagulation took place. At measured intervals during a settling period of 60 min., small samples were withdrawn for determinations of floc concentration. These samples were siphoned out through sampling tubes of $\frac{1}{4}$-inch copper tubing, the inlet ends of which were bent-in horizontally to withdraw the samples from a depth of $2\frac{1}{4}$ in. above the bottom of the jars.

Two methods for the measurement of floc concentration in the samples seemed feasible—the test for turbidity and the test for alumina or iron in the precipitate. Because of the low content of

suspended matter in the floc, the "suspended solids" test could not be used. Since turbidity measurements vary with the size and character of the suspended particles as well as with their concentration and since the method of withdrawal of the samples promised to affect the turbidity, the turbidity test was discarded in favor of the alumina or iron test. A comparatively simple and reasonably accurate method for the determination of iron in waters with low organic matter content is described in *Standard Methods*. As no such simple or reliable method is available for the determination of alumina, iron was selected as the coagulant in these studies. To insure the presence of a maximum amount of iron in the precipitate, ferric iron was selected, and ferric sulfate was chosen for convenience. It is felt that the results of the experiments with the iron salt are of such a nature that the conclusions drawn have general applicability for other coagulants.

The coagulant used was "Ferrisul" which was furnished through the courtesy of the Monsanto Chemical Company, Merrimac Division. Alkalinity was added in the form of sodium hydroxide, and pH adjustment was obtained by regulation of the dose of NaOH. Both the $Fe_2(SO_4)_3$ and the NaOH were prepared as strong, clear solutions of known concentration and were measured into small beakers just before being dosed into the jars. Since both beakers contained only about 50 ml. of solution, the effect of the addition of these solutions upon the temperature of the water in the jars was negligible. In dosing, the NaOH solution was added first and was followed at once with the Ferrisul solution.

pH determinations were made colorimetrically upon small samples removed from the jars. It is worthy of note that the pH measurements made in this manner are sufficiently accurate for control of the experiments, but the values recorded are not to be taken as the actual pH values of the liquid in the jars. The difference in temperature existing between the liquid in the battery jars and the samples withdrawn for analysis was in many cases sufficient to modify substantially the pH reading obtained. For true pH values, measurements should be made electrometrically directly in the jars at the temperature of the test.

To compare settling curves made at different temperatures, it was necessary to adjust them all to the same temperature of settling. The adjusted temperature was arbitrarily taken at 10°C.; and all the settling curves were redrawn to indicate what the removal would

have been had the temperature of the water, after mixing was stopped, been brought instantaneously up or down to 10°C. without in any way affecting the size, shape or weight of the floc particles. Since the time of settling is directly proportional to the viscosity of the water, the adjustment of a curve for any temperature T°C. may be made by modifying the abscissae of points on the curve by the ratio of the viscosities, $\frac{\eta_{10}}{\eta_T}$. The adjusted settling periods will be longer for tests made at temperatures above 10°C. and shorter at

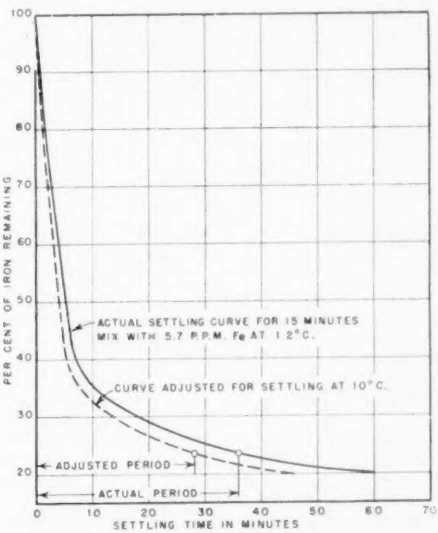

FIG. 2. Method of Adjusting Settling Curves for Temperature Effect

lower temperatures. Figure 2 shows an original settling curve and its adjusted position.

Results

Runs were made with doses of ferric sulfate, $Fe_2(SO_4)_3$, molecularly equivalent to 2, 1.5, 1.1 and 0.7 g.p.g. alum, $Al_2(SO_4)_3 \cdot 18H_2O$. These mixtures contained 5.7, 4.3, 3.15 and 2.0. p.p.m. iron, as Fe, respectively. The mixing periods used for each dose were as follows:

5.7 p.p.m. iron	10, 15, 20, 30, 45 and 60 min.
4.5 " "	15, 20, 30, 45 " 60 "
3.15 " "	20, 30, 45 " 60 "
2.0 "	30, 45 " 60 "

The smallest mixing period used for each dose was just long enough to produce floc which would settle with reasonably consistent results.

All the above runs were made at three different temperatures: 28.3°C., 15.1° to 15.3°C. and 1° to 5°C.

After the elapse of the allotted mixing time for each run, the paddle was stopped and samples were withdrawn for iron analysis at 0, 1, 3, 5, 8, 15, 30, 45 and 60 min. during the subsequent settling period.

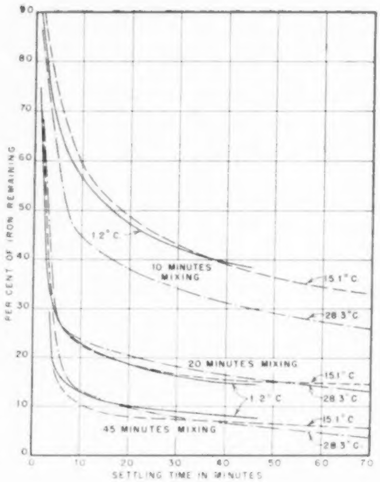

FIG. 3

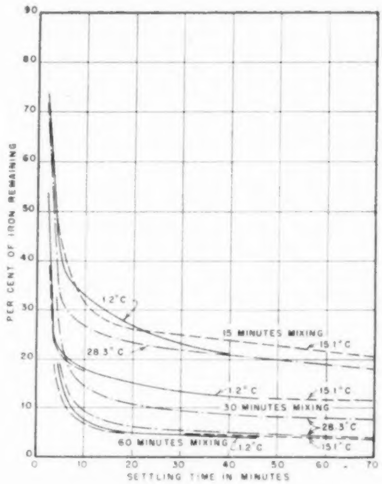

FIG. 4

FIG. 3. Settling Curves Adjusted to 10°C. for an Iron Dose of 5.7 p.p.m.; after 10, 20 and 45 min. mixing

FIG. 4. Settling Curves Adjusted to 10°C. for an Iron Dose of 5.7 p.p.m.; after 15, 30 and 60 min. mixing

Every run was repeated at least twice, so that every curve presented represents the average of at least three independent runs. A study of the deviation of the results for similar runs indicated that the maximum error of any point on an average curve, measured in terms of iron concentration was probably less than 10 per cent. The average settling curves, adjusted for settling at 10°C., are shown in Figs. 3 through 9. To facilitate comparison, the ordinates are plotted as ratios of iron concentration to initial iron concentration.

The optimum pH value for coagulation with an iron dose of 5.7 p.p.m. was found to be about 6.8; and there was no measurable

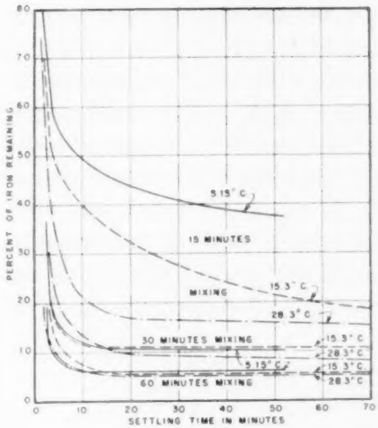

Fig. 5

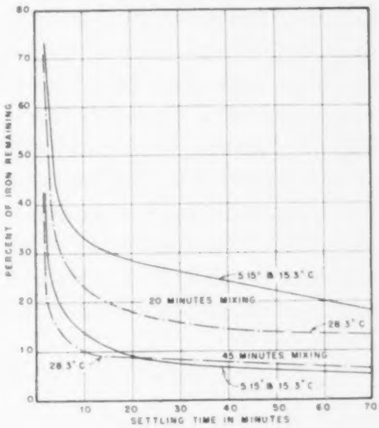

Fig. 6

Fig. 5 Settling Curves Adjusted to 10°C. for an Iron Dose of 4.3 p.p.m.; after 15, 30 and 60 min. mixing

Fig. 6. Settling Curves Adjusted to 10°C. for an Iron Dose of 4.3 p.p.m.; after 20 and 45 min. mixing

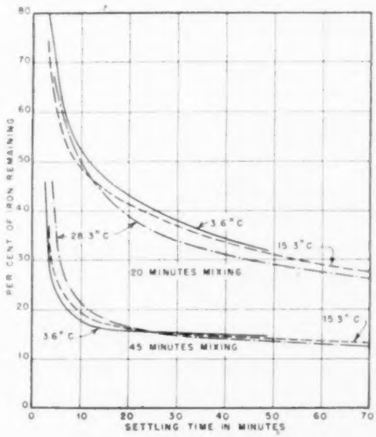

Fig. 7

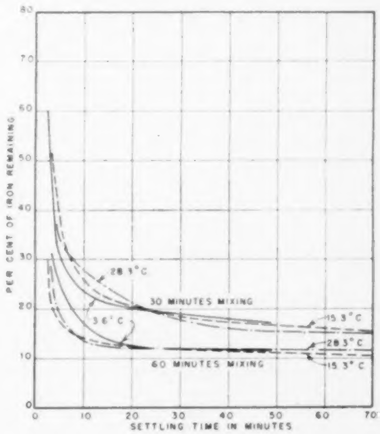

Fig. 8

Fig. 7. Settling Curves Adjusted to 10°C. for an Iron Dose of 3.15 p.p.m.; after 20 and 45 min. mixing

Fig. 8. Settling Curves Adjusted to 10°C. for an Iron Dose of 3.15 p.p.m.; after 30 and 60 min. mixing

variation in this optimum pH value with variation in temperature. For the lower concentrations of coagulant, the optimum pH was found to vary with both the temperature and the amount of the iron dose. The results of the optimum pH studies are indicated in Fig. 10. As has been previously pointed out, the numerical values of the pH determinations are not to be considered reliable because the tests were made colorimetrically on samples withdrawn from the jars. There are errors introduced both because of the change in temperature due to the withdrawal of the samples from the jars and because of the inherent inaccuracy of colorimetric methods on

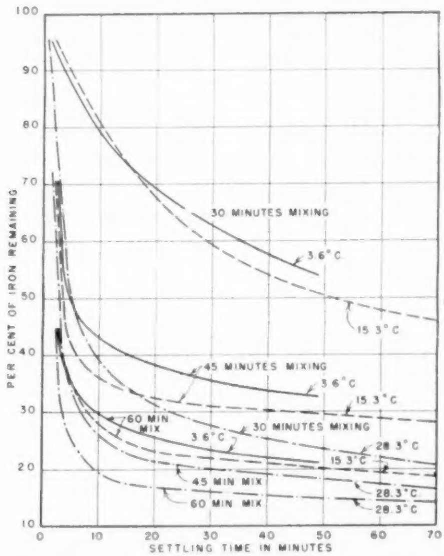

FIG. 9. Settling Curves Adjusted to 10°C. for an Iron Dose of 2.0 p.p.m.; after 30, 45 and 60 min. mixing

lightly buffered solutions. Nevertheless, since the pH determinations were all made in the same way, they may be used safely for comparative purposes. Figure 10 indicates that the optimum pH value increases as the concentration of iron dose is decreased. It also indicates, though not conclusively, that the optimum pH value is higher for the lower temperatures.

Discussion

An inspection of Figs. 3 through 8 indicates that the curves for different temperatures are almost coincident. The lack of agreement

is less than the experimental error in most cases. Rather wide discrepancies are noted for only one case out of the fifteen cases shown. In Fig. 9, for the smallest coagulant dose, the agreement is not so close. All three cases indicate a better floc for the higher temperature, with little difference in floc formed at 3.6°C. and 15.3°C. These three erratic curves may be due to experimental error, as some difficulty was experienced in obtaining close control for the low coagulant dose. Runs were attempted with a dose of only 1.4 p.p.m. of iron, but, in the absence of turbidity or color in the raw water, no consistency could be obtained with so little coagulant.

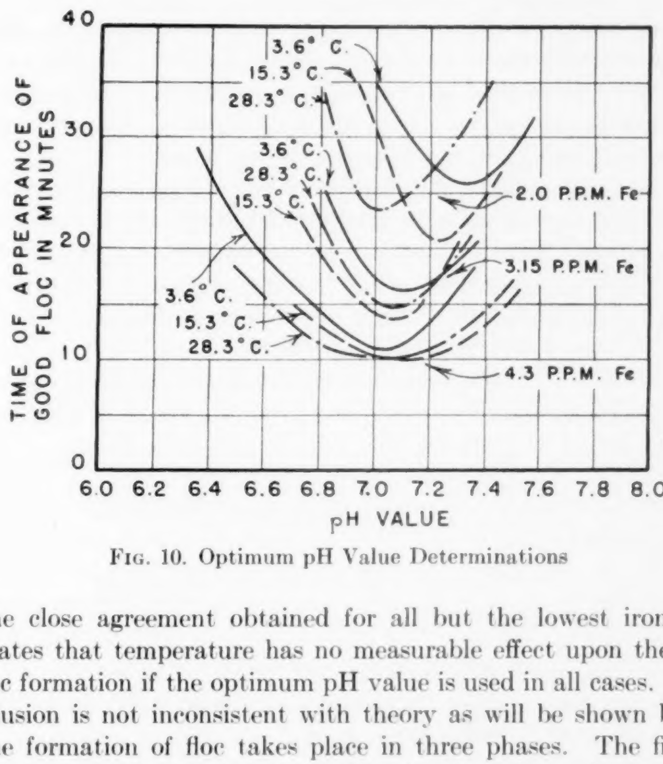

FIG. 10. Optimum pH Value Determinations

The close agreement obtained for all but the lowest iron dose indicates that temperature has no measurable effect upon the time of floc formation if the optimum pH value is used in all cases. This conclusion is not inconsistent with theory as will be shown below.

The formation of floc takes place in three phases. The first of these phases is the formation of molecular precipitates by means of chemical reactions. Temperature does affect the speed of these reactions, but they are so rapid, even at the lower temperatures, that the time required for the formation of the precipitate is of no moment as compared to the time of formation of the floc. For practical purposes, the chemical reactions may be considered as instantaneous.

The second phase in coagulation is the formation of colloidal floc particles. Both the molecularly dispersed particles and the colloidal particles are moved rapidly about due to the Brownian effect or diffusion. This motion brings them in contact with one another, whereupon they coalesce, forming larger particles. As the size or mass of the particles increases, their diffusion velocity decreases until eventually the particles reach such sizes that their diffusion velocity becomes negligible; in other words, they no longer move about under their own kinetic energy. They have reached the upper limit of size of colloidal particles. The size of these particles is, however, still very small—approximately 1 micron, or $\frac{1}{25,000}$ in. in size and barely discernible under the microscope. Hence, further mixing must be applied mechanically to the particles to build them up in size until they are large enough to settle. The mechanical stirring constitutes the third phase of coagulation.

The rate of flocculation in the second or colloidal phase, according to von Smoluchowski (13), is proportional to the velocity of motion of the colloidal particles. This velocity, in turn, is proportional to the "coefficient of diffusion" of the particles in water. Using Stokes' value of the "coefficient of drag" of spheres in laminar flow, Einstein (14) has shown that the coefficient of diffusion is directly proportional to the absolute temperature of the fluid and inversely proportional to the viscosity. Therefore, the speed of coagulation due to Brownian motion is directly proportional to the ratio $\frac{T}{\eta}$, where T is the absolute temperature and η is the absolute viscosity. This ratio has a value, at 30°C. (86°F.), which is more than twice its value at 1°C. In other words, flocculation due to Brownian motion at 86°F. is more than twice as fast as it is near the freezing point of water. Whether this influence of temperature is of importance in coagulation with iron and aluminum salts depends upon the relative amount of time required for completion of the Brownian motion phase of coagulation.

When observations were made, by means of the light beams, of the time of appearance of the floc in the battery jars, a very faint Tyndall cone was noticeable with the distilled water. This cone was unchanged after the addition of the NaOH solution. With the larger iron dose, 5.7 p.p.m., a definite cloudiness appeared in the beam within 6 sec. after the addition of the ferric sulfate at the

higher temperature and within 10 sec. at the lower temperature. The intensity of this cloudiness increased until about 6 min. had passed, at which time the individual floc particles were just barely discernible to the naked eye. The time of appearance of the individual particles was not noticeably affected by the temperature. The Brownian motion phase of coagulation was probably well on its way to completion in the 6 to 10 sec. required for the cloudiness to appear. Since this period is so small, as compared with the time required for the particles to become plainly distinguishable, the practical effect of temperature during the Brownian motion phase upon the whole of coagulation is negligible. At the lower iron doses, the time of appearance of the cloudiness or haze could not be measured with consistency. The time of appearance of clearly discernible particles, however, was little affected by the temperature.

In the third phase of coagulation, the particles are brought together by the motion of the liquid. The speed of coagulation at any point in the liquid is proportional to the velocity gradient at that point. For a particular flow pattern in a tank, therefore, the speed of coagulation is directly proportional to the speed of mixing. If the mixing velocity is high enough to mask the effect of convection currents and produce a stable flow pattern, the speed of coagulation is independent of the temperature. The results of the experiments are in agreement with these relations.

One of the most significant results of the experiments is the finding that the optimum pH value for flocculation changes markedly with variation in concentration of coagulant and is also influenced by temperature variation. These relations are indicated by Fig. 10. The small effects of temperature at the higher coagulant doses are insufficient reason for ignoring the effects indicated at the smaller iron dose. The smaller doses, because of their economy, are the more important to operators.

At many treatment plants where the raw water has ample alkalinity for the chemical reaction, no effort is made to adjust the pH to the optimum point by adding alkali or acid. In such cases, the actual pH used for coagulation may be far from the optimum, as was true in the case reported by Velz (2). The amount of coagulant required for adequate flocculation, in the time available in the mixing chamber, may consequently be much greater than would be required at the optimum pH. This is clearly indicated by the shape of the curves in Fig. 10. If, then, the change in optimum pH with the

temperature of the water is such as to approximate more nearly the actual pH used, less coagulant will be required at the changed temperature. The apparent anomaly reported by Velz may thus be explained as the effect of temperature upon the optimum pH value. Velz, as was previously noted, reported a decided change in the optimum pH of his water between winter and summer temperatures.

The finding that temperature changes cause shifts in the optimum pH value is not unexpected. The equilibrium constants of chemical reactions are, of course, influenced by the temperature. Hence, isoelectric points are similarly affected. It is probable that the flocculating value of ions is also influenced by the temperature, inasmuch as adsorption of these ions is a part of the process of flocculation. There is insufficient information available at the present time to indicate the direction and amount of shift of the optimum pH for a particular water and coagulant. It is, therefore, quite important for economy of coagulation that jar tests for optimum pH be made at the temperature of the raw water and not at room temperature. If this practice is to become general, laboratory stirring devices must be equipped with constant temperature baths.

Conclusions

The following conclusions may be drawn from this study:

1. Changes in temperature have no measurable effect upon the time of formation of floc, if coagulation takes place at the optimum pH value.

2. The optimum pH value is shifted by changes in temperature, and the influence of temperature is greatest for the smaller coagulant doses.

3. The optimum pH value is shifted by changes in the amount of the coagulant used.

4. The greatest economy of coagulant is obtained if flocculation takes place at the optimum pH value.

5. Laboratory jar tests for optimum pH value and coagulant dose should be made at the temperature of the water in the mixing basin.

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Gastro-Enteritis Outbreak Traced to Painting of Water Tank

By M. S. Campbell

ON MAY 11, 1939, it became apparent to the Washington State Department of Health that more than a usual incidence of gastro-intestinal disorder was occurring among the pupils of the Garfield School in Olympia. It was reported that about 35 to 40 pupils were ill on May 10, and that many of these became ill in school.

Outbreaks similar to this are not uncommonly reported in water works and medical literature. In some instances the causative organism and the means of spread are readily determined. Other outbreaks are very puzzling and only assumptions can be made. Nevertheless, careful study is desirable.

Epidemiological, engineering and laboratory personnel of the state department of health, together with local county-city health department personnel, co-operated in this study.

The first indication of an outbreak came on the afternoon of May 9, when the school principal reported to the local county-city health department that ten or twelve pupils had become ill with nausea and vomiting. On Friday morning, May 11, a survey of the pupils was begun—epidemiological histories being taken on about 60 pupils who had been ill. The fact that the school enrollment was only 450 pupils made it clear that there was a true outbreak. Its rather explosive character may have indicated food poisoning, possibly through means of the school lunch room. The investigation as concluded on May 17, however, revealed that only 17 of the 76 pupils who were ill had bought any part of their lunch at the school lunch room. Milk sold was pasteurized and bottled and was bought from three different dairies. None of the other schools in Olympia using

A paper presented on May 10, 1940, at the Pacific Northwest Section Meeting, Portland, Oregon, by M. S. Campbell, Assistant Public Health Engineer, Washington State Department of Health, Seattle, Wash.

milk or milk products from these dairies reported any gastro-intestinal upsets.

A house-to-house canvass in the neighborhood of the school supplied by the same water system revealed that eleven adults and two pre-school children were also ill. All the physicians in Olympia, as well as school and health authorities, were on guard for new cases. Some were reported, but all were already included in the number reported after the house-to-house canvass. Only one of the adults had been to the Garfield school within the ten days previous to his illness. It was considered significant too that all of those ill had their residences on the same water system that supplied the school, and that not all of the ill adults had children in school.

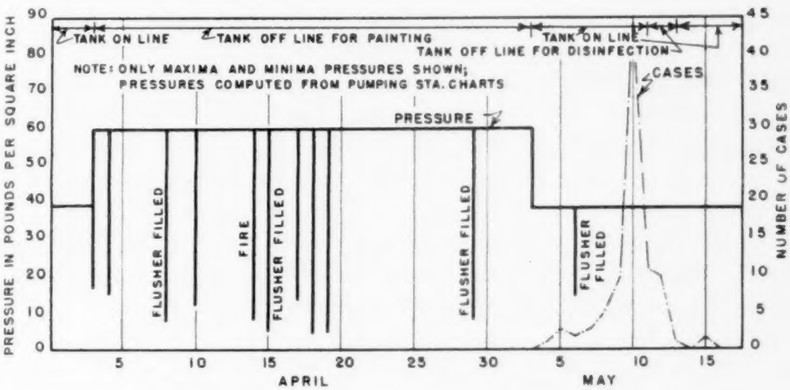

FIG. 1. Relationship of Outbreak to Painting of West Side Tank and to Pressures in Distribution System at Garfield School Street Main

The clinical findings were few—only six of those who were ill had any fever. The symptoms were quite typical with a rather sudden onset of nausea, slight vertigo and vomiting. Most of them did not feel ill enough to go to bed, but had only a feeling of fatigue and thirst. Some significance, too, was attached to the fact that of the 76 children and 3 teachers ill, 57 became ill at home and 22 became ill at school. Most of them awoke feeling nauseated between 2 A.M. and 6 A.M. On May 10, the day of the outbreak, 44 became ill. Of these, 28 became ill at home in the early morning and 16 became ill at school, with most cases occurring in the early afternoon. Some became ill so suddenly that they were unable to reach the lavatory before vomiting. The illness was of rather short duration, averaging

about 18 hours and ranging from 6 to 48 hours. No serious effects have been noted.

Of the 76 pupils ill, 57 vomited and 16 had diarrhea. Of the 14 adults found ill there were 6 with vomiting and 8 with diarrhea, with 6 adults having both vomiting and diarrhea. The two pre-school children ill had no diarrhea but vomited from three to six times. All cases indicated an irritation of the upper end of the gastro-intestinal tract, which would be typical of a toxic poisoning rather than bacterial infection.

The peak of the outbreak, as shown in Fig. 1, occurred on May 10. The first case appeared on May 4, with only a few occurring each following day until May 9, when 10 became ill; and finally the outbreak of 44 came on May 10. The next two days, 11 and 10 cases, respectively, occurred, and after May 15, no cases were found.

Of the 92 found to be ill, 80 were school children and adults who had been drinking the Garfield School water during, and within ten days of, their illness. The remaining 12 were all on the same water system supplying the school. Figure 2 indicates the location of the cases with reference to the Garfield School and the elevated water storage tank.

Bacteriological laboratory findings were significantly negative. Water samples showed no indication of bacterial contamination. Vomitus and stool samples showed no pathogenic bacteria.

Public Health Engineering Findings

The epidemiological investigation revealed that the one factor common to all of the cases was the water supply. It was highly significant that all of those who became ill either attended the Garfield School, which is supplied with city water, or lived in houses which had a city water service connection from the distribution system in that vicinity.

The city of Olympia is adequately served by a municipally owned water system derived from artesian wells. The flow from the wells is by gravity to the main pump station where the supply is disinfected with ammonia and chlorine. The supply is then pumped into two open concrete reservoirs, each of 2,500,000 gal. capacity, and three elevated steel tanks, one of 200,000 gal. capacity and two of 250,000 gal. capacity. Olympia is so situated that Budd's Inlet divides the city into what is known as the East Side, which includes the business district, industrial area, State Office buildings, and residential area,

and the West Side, a residential area located on a hill overlooking Budd's Inlet. The locale of the outbreak was on the West Side, which is supplied with water from the distribution system on the

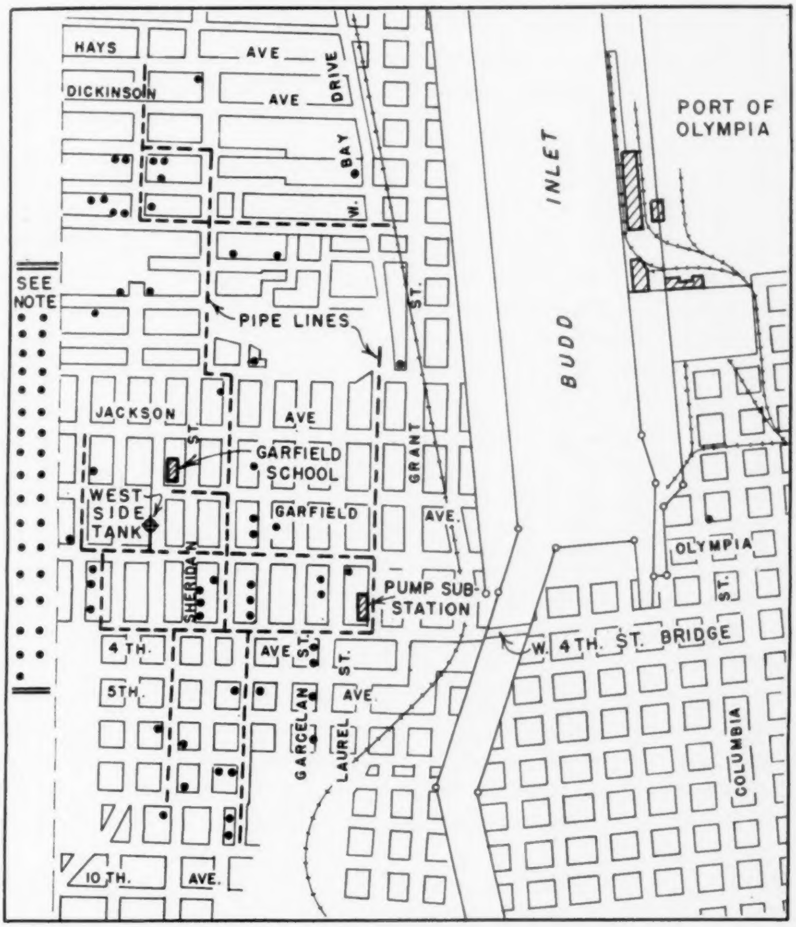

FIG. 2. Vicinity of Outbreak; Showing Geographic Distribution of Cases. NOTE: Cases Residing in Rural Area and Attending Garfield School

East Side by a line across Budd's Inlet on the West Fourth Street Bridge. A booster pump station, located at Fourth Avenue and West Bay Drive, supplies the West Side and the 200,000 gal. elevated steel tank which floats on the distribution system. The West Side

booster station is equipped with three pumps having capacities of 200 g.p.m., 500 g.p.m., and 680 g.p.m., pumping against a head of 205 ft. The pumps are automatically controlled by a pressure actuated mechanism in such a manner that, as the demand increases, additional pumps begin functioning. The chlorination plant reports indicated that the disinfection equipment was operating normally prior to and during the outbreak. This was substantiated by con-

TABLE 1

Results of Bacteriological Analyses

DATE	SAMPLING POINT		COLIFORM ORGANISMS PER 100 ML.
4- 4-38	Settling tank	(raw)	0
4- 4-38	Reservoir No. 2	(treated)	0
5-18-38	Standpipe	(")	0
7-25-38	East Side Reservoir No. 2	(")	0
7-25-38	Pump house	(")	0
7-25-38	Settling basin	(raw)	0
8- 3-38	Tap at City Hall	(treated)	0
8- 5-38	Standpipe	(")	0
8-22-38	Reservoir No. 2	(")	0
8-22-38	Pump house	(")	0
8-22-38	Settling tank	(")	0
11-20-38	Reservoir No. 2	(")	2
2-27-39	Reservoir No. 2	(")	0
2-28-39	Pump house	(")	0
2-28-39	Settling basin	(raw)	0
5- 1-39	Reservoir No. 1	(treated)	0
5- 1-39	Settling tank	(")	0
5-10-39	West Side tank	(")	0
5-11-39	Hydrant at End of Decatur St.	(")	0
5-11-39	Hydrant at End of Decatur St.	(")	0
5-11-39	Thomas & Hayes St. Hydrant	(")	0

ference with the operator. The results of the bacteriological analyses of the Olympia supply for the period from April 4, 1938, to May 11, 1939, are shown in Table 1.

In the course of the investigation a conference with the city engineer revealed that the elevated steel tank (hereinafter referred to as the West Side Tank), located in the immediate vicinity of the Garfield School, had been painted on the inside and placed back in service just previous to the outbreak. Subsequent investigation revealed that the tank had not been disinfected after completion of the work and prior to being placed in service.

Painting Procedure

Painting of the tank was carried out as follows:

At noon on April 3, the tank was drained for the purpose of painting. Painting was concluded at 4 P.M. on May 2. On May 3, between 8 A.M. and 12:30 P.M., the inside of the tank was washed down with water from the distribution system, without, however, washing down the inside of the tank vapor space. At 12:30 P.M., May 3, the tank was cut back on the line, and at 9 P.M., the same day, was completely filled. On May 4, the following day, the first case of the outbreak developed. Figure 1 shows graphically the relationship between the outbreak and the time of placing the West Side tank in service after painting. Immediately upon learning of the circumstances surrounding the cleaning and painting of the tank, it was cut out of service at 8:30 P.M. on May 11, and drained. At 9:45, the tank was filled with heavily chlorinated water and was, then, allowed to stand out of service until the morning of May 13, completely filled with water having a residual chlorine content of approximately 20 p.p.m. On May 13, the tank was returned to service.

Although suspicion was directed at the tank, the Garfield School Building was checked for the possibility of cross-connections. Two possible sources of cross-connections were found: one being the inlet pipes to several automatic flushing urinal boxes, located in the main building, and which extended several inches below the surface of the water in the reservoirs. The elevation of the flush boxes was such that if the pressure dropped to approximately 7 lb. per sq.in. in the main supplying the building, back siphonage would take place and result in siphoning the contents of the boxes into the water system. (This was subsequently found to have occurred.) Although these boxes were uncovered there was no evidence to show that back siphonage from them would have had any bearing on the outbreak.

Another possible cross-connection in the school building was between the drain, from the sump into which the boiler drained and from which an automatic sump pump lifts the drained boiler waters into the sewer, and the sewer located in the building. Had sewage overflowed from the sewer into the sump, a possible cross-connection would have existed. Tests, however, indicated that the design of the building sewer was such that even under conditions of extreme overload, sewage would not pass over into the sump. With the possibility of cross-connections as the cause of the outbreak eliminated by both the epidemiological study and the cross-connection investigation, the interest centered entirely upon the West Side Tank.

In view of the fact that the epidemiological investigation had shown that the outbreak was not due to a bacterial infection, suspicion pointed directly to a chemical poisoning resulting from the painting of the tank.

The painting specifications required: (1) that the existing enamel be removed by chipping, followed by shot blasting; (2) that all surfaces to be covered with enamel be cleaned by washing with a coal-tar solvent (crude solvent naphtha); (3) that the priming coat, composed of a coal-tar pitch base and crude solvent naphtha, be applied, allowing not less than 24 hr., nor more than 96 hr., for drying before the application of the hot enamel; (4) that the enamel, composed of coal-tar pitch with a siliceous mineral filler, be, then, applied at a temperature of not less than 485°F., nor more than 500°F.; and (5) that the enamel coat be spark-tested for thickness, and in those areas not meeting specifications, additional enamel be applied without either shot blasting or adding more primer.

The temperature of application of the hot enamel was checked by an automatic recording thermometer.

Possible Causes

Sifting of the evidence eliminated all but the following four possibilities which might have been responsible for the outbreak.

1. The possibility of the entrance of either old enamel, when it was removed, or primer paint into the riser pipe leading to the tank. This possibility was eliminated due to the fact that the riser extends four feet above the bottom of the bowl and was equipped with a blind flange, making it entirely water tight. The riser was full of water under pressure up to the blind flange during the painting operations. In addition, a hand hole, through which all foreign matter reaching the bottom of the tank was flushed, is located in the bottom of the bowl next to the riser.

2. The possibility of priming of the ladders, spreader rods, etc., without the enamel coat, thus making it possible for water to come directly in contact with primed surfaces when the tank was filled. Since the ladder was enameled, as was the inside of the shell, and no primer was applied to the spreader rods, this possibility was eliminated.

3. The possibility of primer being spilled in the tank. In view of the rumor, which was repeated to the writer on various occasions, that a bucket of primer was spilled in the tank after enameling was complete, and the fact that this version apparently originated among

the "hot-enamel men," this possibility was seriously considered. Assuming that a bucket of primer actually entered the contents of the tank, however, the quantity would hardly be sufficient to produce the widespread outbreak that occurred. Furthermore, the tank was washed down and flushed out through the hand hole in the bottom of the bowl immediately prior to filling. It appears, therefore, that even had the primer been spilled that insufficient residual would have remained to warrant further consideration of this possibility. A bucket of enamel was spilled in one of the other two tanks when the staging collapsed and this perhaps resulted in the rumor that primer was spilled in the West Side Tank.

4. The only possibility, impossible of elimination, was that volatile portions of the priming coat, containing toxic substances, by some method came in contact with the water after enameling was complete and the tank filled. In reconstructing this possibility the following picture presented itself. The priming coat was applied to the entire inside of the shell of the tank before any enameling was done and at least 24 hr. were required to elapse between application of the priming coat and the enamel coat. During this time some of the volatile portions of the priming coat left the inside of the shell and were condensed on the inside of the tank in the vapor space. The enamel was then applied to the inside of the shell and the bottom of the tank at 485°F. to 500°F. Any of the volatile portions of the primer which had not evaporated during the 24-hour drying period would, during the application of the hot enamel, be volatilized and would also condense on the inside of the vapor space.

The inside of the vapor space of the tank was not painted at this time because it had been painted two years before. The other two steel elevated tanks in the system had been painted just prior to the painting of the West Side Tank, but in both of these tanks, aluminum paint was applied to the spreader rods and to the inside of the tank in the vapor space. The volatile portions of the primer which had been deposited on those surfaces in the other tanks had thus been removed. The volatile portions of the priming coat which had evaporated in the shell and condensed on the inside of the vapor space in the West Side Tank had no possibility of escape except through one small hatch in the roof. Consequently, when the tank was placed in service the volatile portions of the primer must still have been present on the inside of the tank in the vapor space, and as a result of condensation of water vapor, must subsequently have been carried back into the contents of the tank.

Analyses of Enamel and Primer

Samples of the primer and enamel were submitted to Laucks Laboratories, Seattle, for analyses, to determine the presence or absence of toxic substances. Their report on the enamel based upon spectrographic analysis, follows:

"The absorption spectrum on the sample of asphaltic enamel base fails to reveal the presence of toxic bases such as benzenes, zylenes, etc. This test is not of the value that it should be, because the asphalt has tended to absorb in the areas commonly used in identifying the lighter fractions.

"While this test is not conclusive, it is our opinion that the asphaltic enamel base does not contain harmful or injurious ingredients to any great extent."

The report on the analysis of the primer, based upon fractional distillation, follows:

"We hereby certify that we have tested the sample of paint primer submitted to us by you, and we have to report as follows:

"Fractional Distillations:

Up to 335°F.	4.4	per cent
From 335°F. to 346°F.	11.9	"
From 346°F. to 350°F.	6.1	"
From 350°F. to 375°F.	10.3	"
From 375°F. to 410°F.	5.3	"
Residue not distilled	62.0	"

"This distillation shows that this primer did not consist of a heavy asphalt or tar, thinned with a light solvent. This distillation shows it to be a blend of a comparatively soft tar and a comparatively heavy solvent. The residue which we obtained in our distillation consists of a material as hard as asphalt and having somewhat the same odor as asphalt. However, the last fraction that came off between 375°F. and 410°F. consisted almost entirely of naphthalene. We did not try to establish whether this was a petroleum naphthalene or a coal-tar naphthalene. We did test it to see whether or not it contained phenol and found no trace of phenol present. This, of course, would indicate that it was probably a petroleum naphthalene.

"Because the fractions from this primer had such high boiling points we are inclined to believe that the primer would be rather slow to dry and any attempt to apply a hot enamel over it would

necessarily produce fumes which of course would condense on the first cold surface with which they came in contact.

"Naphthalenes are commonly used as disinfectants and insecticides and produce an irritating action on the digestive tract when taken internally. Therefore, we believe that if water were allowed to become contaminated with any great amount of this material, the water would be toxic to human beings."

Contributory Operating Conditions

Under ordinary operating conditions, with the West Side Tank floating on the line, the demand from that section of the distribution system would be supplied largely from the pumping station with only small quantities of water being withdrawn from the tank at intervals. According to this hypothesis, only relatively small amounts of the primer could possibly have been present, and since only small quantities of water apparently would be withdrawn from the elevated tank at intervals an explanation of the explosive nature of the outbreak was lacking. It would have been necessary for a heavy draft to have been placed on the elevated storage tank in order to distribute the contents from this tank in sufficient concentration to produce an outbreak as explosive in nature as this one. With this thought in mind, the pressure conditions existing in the distribution system during the time the tank was out of service were investigated. The heaviest drafts which could have occurred on the West Side distribution system would have been during fires and the filling of the street flusher. The times and location of fires prior to the outbreak were as follows:

April 14—Brush fire; hydrant at 5th and Garcelan; 12:32 P.M.

April 21—Brush fire; hydrant at 5th and Garcelan; 12:48 P.M.

May 13—Old McCleary Mill; hydrant at Hays and W. Bay; 1:20 P.M.

May 14—Old McCleary Mill; hydrant at Hays and W. Bay: 8:15 A.M.

The street flusher is always filled from a hydrant located at West Fourth and Laurel Streets by opening the hydrant four turns or the equivalent of one-fourth open. The times of flusher fillings were as follows: April 8, 8:00 A.M.; April 15, 8:00–9:00 A.M.; April 29, 8:00 A.M.; and May 6, 8:00 A.M. According to the city engineer, the draft resulting from filling the flusher is the equivalent of a draft produced by one fire hose.

Of major importance is the fact that during the period the West Side Tank was out of service for painting, the entire West Side distribution system was fed from one 200-g.p.m. pump, although a 500-g.p.m. and a 680-g.p.m. pump were available at the pump station. During the time the tank was out of service, the automatic pump control equipment was not functioning and the pumps were operated manually. Therefore, in order to check the pressures, a gage was placed on the fire hydrant next to the Garfield School on the corner of Sheridan and Garfield Streets. It was found that under normal operating conditions, with the tank on the line and no pumps operating at the booster station, the pressure remained at 40 lb. per sq.in. with the hydrant at West Fourth and Laurel Streets open four turns. With the elevated tank off the line and the 200-g.p.m. pump operating against the closed valve, the pressure at the corner of Sheridan and Garfield Street was 60 lb. per sq. in. Under this condition, however, when the hydrant at West Fourth and Laurel Streets was open four turns (the equivalent of the draft obtained when the flusher was being filled), the pressure at Sheridan and Garfield dropped to zero pounds within one minute. In view of these results, only one conclusion could be drawn—that with the tank on the line, and the same draft imposed on the distribution system, practically the entire demand of the West Side distribution system was being supplied directly from the elevated tank, undoubtedly in sufficient quantity to have carried the contents of this tank well into the distribution system within a very short period of time. A heavy draft such as this, placed directly on the tank, can well account for the explosive nature of the outbreak. The fact that the flusher was filled on May 6, with a resulting drop in pressure to 14 lb. in the distribution system just prior to the peak of the outbreak, contributes the final evidence regarding the cause of the 92 cases of gastro-enteritis.

Conclusions

1. Bacteriological analyses of the supply before and during the outbreak did not indicate contamination as measured by *Standard Methods of Water Analysis*.

2. The purification process was functioning normally before and during the outbreak.

3. The West Side Tank serving the locale of the outbreak had recently been painted on the inside and returned to service without washing down of the inside of the vapor space.

4. While the specific etiological agent of the outbreak is unknown, evidence uncovered points accusingly to a toxic substance or substances contained in the primer paint.

5. The evidence clearly indicates that by evaporation of the primer and subsequent transfer to the contents of the tank, by condensation of water vapor in the vapor space, the toxic volatile portions of the primer gained access to the water supply.

6. Excessive drops in pressure in the West Side distribution system were permitted to occur because of the fact that, while the West Side Tank was out of service during the painting operations, a 200-g.p.m. pump was depended upon to serve the entire area, even though two additional pumps of 500 and 680 g.p.m. capacity were available at the pumping station. These pressure drops would have made back siphonage in the distribution system possible.

7. Although the 92 cases of gastro-enteritis reported may be considered a major outbreak of this character, the fact must not be overlooked that a water-borne epidemic of more serious consequences, as a result of back siphonage during periods of drop in pressure, was averted only by good fortune.

8. Since washing, flushing, and disinfection of the tank in the inside of the vapor space appears to be as important as cleaning the remainder of the tank, this procedure should be included in the painting specifications and be made a part of the regular procedure of all operators of water supplies.

9. Paint manufacturers supplying water works paints and enamels should take suitable precautions to guard against the use of toxic substances in their products.

10. When elevated tanks are cut out of service for any reason, sufficient pumping capacity should be available to eliminate excessive pressure drops in the distribution system and/or suitable steps should be taken to eliminate the causes of excessive drafts on the distribution system wherever practical.

Protective Grounding of Electric Installations

Present Attitude of A.W.W.A.

By Harry E. Jordan

THE propriety of the use of water service pipes for the purpose of protective grounding of electrical installations has been discussed since the practice began about thirty years ago. When the permission to use water pipes for this service was first requested, it was represented that there would be no current interchange over these connections except at the infrequent intervals when they were exercising their protective functions. It is now demonstrated that, in many cases, from 25 to 75 per cent of the normal alternating current load travels through the grounding connection and over the water service pipe.

When the 1940 Electrical Code was presented for approval to the A. W. W. A. representative on the A. S. A. Council, its phraseology on the topic of grounding appeared to require a statement concerning the attitude of this Association upon the subject. This statement was made by letter dated August 20, 1940.

A number of questions are involved in the discussions which have now been carried on for thirty years. Why is protective grounding grounding necessary? What are the real features of electric service that require grounding protection? Could it be true that protective grounding is at least in part advisable because the installation and maintenance of electric service facilities are in themselves not as safe as they could be made to be? Is not the fact that a substantial part of the regular current load travels over a ground connection evidence that the electrical installation is defective?

Another category of questions covers the types of grounding. What are the various means by which protective grounding can be effected? Why has the primary emphasis been placed upon the use of water service pipes? Why has "automatic safety switching" not been applied in American practice as it has been in Australia,

England and Continental Europe? Is it possible that water service lines are advocated as grounds simply because they offer the easiest way for the electric industry to dispose of one of its problems?

A third group of questions covers the field which has occupied much of the thought of the Grounding Research Committee. Does the protective grounding of alternating current installations on water service lines damage the water pipes? Does it set up conditions which are dangerous to the safety of water works employees? Does it affect the quality of the water supplied to the consumers? There has been a tendency to examine these questions with an attitude that would require water purveyors to prove damage, danger or consumer complaints to the satisfaction of those who wish to use the water lines for purposes foreign to the distribution of water. Is it not reasonable to suggest the burden of proof is with them?

The final group of questions lies in the legal field. They are important because the electrical code, by state or municipal adoption, becomes the basis of building codes and inspection requirements. The first question then is—who is responsible for the installation of grounding connections? Once these connections are made, who is responsible for their maintenance? Whose business is it to see that they remain useful for whatever service they properly provide? If the water service line is used for protective grounding, is there being created a legal assumption that the water department is obligated to provide such service? Is there a possibility that the water works field may be forced to continue to use materials and methods facilitating grounding when its own interests indicate the use of newer and more useful materials? Finally, if water service lines are used for protective grounding, should not their use be permitted only when the electric utility has applied for the privilege and exercises it only under a contractual relationship which clearly defines its responsibility to the water department for conditions which result?

The variety of questions arising out of electric grounding practices is evident. Doubtless others exist. The answers to them must come from many sources. The electric industry, the water supply industry, the Joint Research Committee on Grounding, the legal minds of the country—all have a responsibility related to the questions involved. And partly for the purpose of indicating that some questions had not been settled, the following communication is on record with the American Standards Association.

August 20, 1940.

American Standards Association,
29 West 39th St.,
New York, N. Y.

Attention Mr. P. G. Agnew, *Secretary.*

Gentlemen:

1. This communication is intended to record at further length the comments made by the undersigned when Letter Ballot 575 was returned on July 30, 1940. This ballot was one of the routine series relating to approval of A. S. A. Standards by members of its Council. It referred to the *1940* edition of the *National Electrical Code* which was prepared under the sponsorship of the National Fire Protection Association.

2. The ballot was returned with the following statement:

"Inasmuch as the opinion is held and has been recorded that the interest of the American Water Works Association does not appear to have been adequately considered in the preparation of this document, I shall neither approve nor disapprove it.

"In particular, the Association's Board of Directors holds the opinion that, while the practice of grounding electric service lines on public water supply service lines is in effect in many cities—water departments and companies do not place the connections; derive no benefit from them; may be damaged by them; and, tolerate them only because of their reputed importance in providing electric service. The water supply industry does not accept responsibility for providing the means of disposal of stray or waste electric current nor does it recognize any legal right of other utilities or services to impose upon its property and services the function of disposing of stray or waste electric current."

3. Over a period of years, the American Water Works Association has observed the development of prior editions of this code and the regulations related to protective grounding therein contained. Its attitude has been based upon a desire to cooperate in the development of reasonable safety measures, provided that these measures did not appear to affect adversely water supply structures or service. Suggestions have been made that the A. W. W. A. be permitted to

join formally in the development of the code along with the eighteen other national and one single municipality groups represented. As an alternate to such representation, through the constructive acts of Doctor Agnew, the American Research Committee on Grounding has been set up and the A. W. W. A. participates in that work.

4. In an abstract of the report of the American Research Committee on Grounding published in the Journal of the A. W. W. A. for September, 1939, there are found the following statements:

> "The question whether alternating current on water pipes does or does not have a detrimental effect on the pipes or the water within them is still undecided and while the Committee has no positive proof that such harmful effects do occur, this lack of evidence does not prove that the action cannot take place.
>
> "The Committee has verified the fact that even under normal operating conditions ground connections from the neutral conductor of electric light or power systems to water pipes regularly carry substantial alternating current ranging from 25 to 75 per cent of the house load. This current interchange usually takes place over the water service pipe between the point of attachment of the ground wire and the street main and over sections of the street main. In a number of cases, however, the Committee has found substantial stray alternating currents on house pipes inside of buildings because the house conduits, cable armor, metal lath, and water pipes afforded a better electrical path than the ground connection.
>
> "The Committee has found a number of cases of electric shock, and one case of a serious fire resulting from radio ground connections. It has found that the ground clamp or ground connection attachment problem requires drastic review both from the electrical as well as the water works viewpoint."

5. The American Water Works Association notes that the research has shown that the travel of electric current on water lines used as a ground is more than adventitious. The development of this fact is in itself highly significant. The Committee's research has definitely made it untenable to assume or to state that current flows over a protective grounding connection only occasionally. It has on the contrary definitely shown that, in the present state of installation practice and with multiple grounding connections installed on water service lines, a definite proportion of the current used by

the householder or other electric consumer is disposed of via the water pipe. It also notes that studies of water service lines and water supplied to consumer, while not indicating that adverse effects should be uniformly anticipated, do indicate that conditions may be set up by grounding which will affect adversely (a) water piping, (b) employees of water departments engaged in their routine duties (c) water customers and/or (d) water delivered to consumers.

6. It is noted that Section 2581 of the 1940 Code (and the corresponding section of previous codes in effect), under the heading "Grounding Electrodes" states:—"*Water Pipe.* A continuous metallic underground water piping system shall always be used as the grounding electrode where such piping system is available."

It is noted that Section 2582 indicates that "when such a water piping system is not available, the grounding connection shall be made in a manner to secure the most effective ground." The Section thereupon lists "(a) metal frame of the building; (b) a gas piping system; (c) a local metallic underground piping system, metal well casing and the like; and, (d) an artificial ground whose electrode consists of driven pipe, driven rod, buried plate or other device approved for the purpose . . . any one or a combination of which may be used for grounding."

7. It appears proper to call the attention of water works men, through publication in the columns of the A. W. W. A. Journal, to the fact that the National Electrical Code for 1940 (as the predecessor editions) does not exclusively require that protective grounding be effected via water service pipes; that they are to be used only when *available*; and that other forms of protective grounding are outlined in the code to be used if and when the water service pipe is not available.

It further appears proper to advise water works men that the *availability* of a water service line as a protective ground may be understood not alone to be based upon the physical fact of its existence but also upon the attitude of the water department toward such grounding practices expressed in the form of properly adopted regulations. It does not appear that the code stipulates the use of water service pipe as the sole possible means of grounding and it will be so interpreted by this Association. As in previous communications upon this subject, water works men will be advised that they should be fully mindful of their rights as well as their responsibilities in connection therewith. At the same time they are advised of their

evident rights in protecting water systems and services against damage, they will be advised to take no steps to remove protective grounds from water service lines without being supported in such acts by legal advice or regularly promulgated regulations or ordinances.

8. The attitude of the American Water Works Association concerning the use of parts of the public water supply system as means of protective grounding may be summarized as follows:

Water works men wish to cooperate in all well considered activities looking toward the protection of the public.

They do not recognize the right of any other public service group to make use of the water system as an incident to the rendering of that service, if the act thus contemplated appears to damage the water supply structures or service.

They hold the opinion that it is incumbent upon the group wishing to make use of the water supply facilities for purposes foreign to the purveying of public water supply, to demonstrate that the acts contemplated or performed do not or will not damage water supply structures, services, employees or customers.

Finally, they do not admit the existence of a general legal right of other utility services to make use of the water system for purposes foreign to the conduct of essential water supply service.

Very truly yours,

Signed

HARRY E. JORDAN,

Secretary.

Progress Report

American Research Committee on Grounding

THE annual meeting of the American Research Committee on Grounding was held in New York on June 29, 1940. At this meeting the Technical Sub-Committee reported on progress made during the past year. This report reviewed investigations made of complaints brought to its attention by representatives of the various organizations associated with the committee, fundamental laboratory studies of polarization, and other grounding problems.

Reports were made on a total of 21 cases which were investigated, representing, however, studies in only seven separate communities. This results from the fact that in connection with most of the complaints, the sub-committee investigated conditions in adjacent and nearby residences to develop more clearly any differences which might account for the variations.

Field Investigations

In one situation, involving a complaint of blue stain, studies were made of the conditions in five separate houses, all of which were built within the last year or two and which were more or less alike in construction and equipment. They all had copper water service pipes from 30 to 50 ft. in length, and the interior piping was of copper. There was definite interchange of current over the water system in three of these cases. The principal differences were the use of copper hot-water tanks in three cases, and galvanized iron hot-water tanks in the other two. Three of the houses, one of them with a galvanized tank and the others with copper tanks, obtained water from a 6-inch cast-iron main within about 100 ft. of its end.

Submitted by H. S. Warren, 420 Lexington Ave., New York, N. Y., Chairman, and C. F. Meyerherm, Secretary, American Research Committee on Grounding, and President and Engineer, Albert F. Ganz, Inc., 511 Fifth Ave., New York, N. Y.

The other two, one each with a copper and a galvanized hot-water tank, were at the dead end of another 6-inch cast-iron main one block away. These two subsidiary mains received their water at points less than 200 ft. apart on a 12-inch feeder main. The water supply was derived from two separate deep-well sources, several miles apart, at the opposite ends of the feeder main. The location of all these houses in relation to the sources was such that they would probably receive water from one source during part of the day and from the other source during the remainder of the day. One of the sources was treated with chlorine but not with lime, while the other was treated with lime but not with chlorine.

There is some evidence which would indicate that deep-well waters may, when they are soft and high in free CO_2, if not treated with lime, cause initial corrosion of copper tubing which would result in blue staining. These conditions may become more serious when the hot water is heated to too high a temperature. In one of the houses, fed at the end of a dead-end main, considerable trouble was experienced from rust and dirt in the water. This was definitely attributed to typical dead-end main conditions which have been found from time to time in the sub-committee's investigations.

The sub-committee reported on an investigation, made in the Middle West, where blue water was involved, the condition being so serious as to cause discoloration of food cooked in the water. In this case the local authorities had called upon the power and telephone companies to remove all grounds from the water piping in the residence, and consideration was being given to extending this prohibition. All the grounds were removed, except the one from the frame of the motor driving the oil burner. The condition was reported to have cleared up temporarily following the removal of the ground, but it recurred within about a month. At that time the house piping was thoroughly cleaned, after which the condition finally cleared up.

During the interval between removal of the grounds and flushing of the piping, a change has been made in the treatment of the public water supply which reduced its iron content from about 2.2 to about 0.3 parts per million. This treatment of the water supply had been discontinued temporarily during the summer months. The house had been completed and first occupied in about the middle of the summer. The conditions in this residence were investigated thoroughly and investigations were made at several other residences similarly equipped but where no complaints had been made of water

conditions. Most of the other residences had been built for some time, in contrast to that where the complaint originated. In connection with the investigation, the sub-committee also made arrangements to have the grounds restored one by one and to have analyses made of the water by the State Health Department at 2- or 3-week intervals to determine whether there was any recurrence of the condition complained of. At the time of the meeting of the Grounding Committee, none of the grounds had been replaced. Advice received later, however, showed that the grounds have been reconnected for some time and that no adverse effects have appeared.

In one case, involving a relatively new suburban house, a serious complaint of blue sediment in the water was brought to the attention of the sub-committee. At the time of the investigation a glass of water drawn from the pipe showed substantial blue flakes of copper carbonate at the bottom. The water service pipe was about 250 ft. long and the electric service was about 50 ft. long from the transformer to the residence. The electric service was grounded at the meter to the water service in approved fashion, and average interchange of current over the water service pipe was revealed. A modified fish trap with an 80-mesh strainer was installed at the house end of the water service to reduce the trouble, and to permit periodic determination of the amount of sediment collected. The amount of sediment fell off to nothing between the late summer and December of 1939. Up to the time of the meeting, no sediment was noted, but in the latter part of August, 1940, sediment reappeared. In this case the water supply was treated with chlorine and lime, the latter by means of an unattended automatic liming equipment at the pumping station. It was reported that the lime treatment had been discontinued since June, 1940.

In this case a section of the service pipe was removed for examination and analysis. The interior of the pipe was found to be coated with non-adherent copper salts. It is of interest that at the time of the removal of this sample of the service pipe, the plumber reported sparking and electric shock. A subsequent investigation revealed that opening of the service pipe resulted in the development of a maximum of 10 volts between the house end and the street end of the piping during the starting of an electric washing machine. With a steady load of 10 amperes in the residence, the voltage across the open ends of the service pipe was 3 volts. This undoubtedly accounts for the sparking and electric shock reported.

The sub-committee, in its report, again pointed out the unsatisfactory conditions which have been observed at the point of attachment of ground clamps to water pipes. Two general types of ground clamps have been observed, one, the copper-strap type, and the other, a cast- or wrought-iron clamp, generally galvanized. The use of copper straps on galvanized pipes sometimes has led to corrosion of both the strap and the piping in the presence of moisture, due either to dampness in the cellar or condensation on the pipe during warm, humid weather. The moisture sets up a local electrolytic cell between the different metals present. Iron clamps on galvanized piping do not appear to have resulted in any adverse effects. The use of large cast- or wrought-iron clamps on copper tubing or on lead pipe, however, has been a source of trouble due to electrolytic effects, and also mechanical damage to the pipe because of the massiveness of the clamp in comparison to the tubing to which it is attached. Improper installation of such clamps may result in crushing or deforming either copper tubing or lead pipe. The sub-committee has discussed the development of a type of grounding connection which could be placed directly in the plumbing system at the time of installation, and to which all electrical ground connections could be solidly made without any of the difficulties which have been observed with clamps.

Fundamental Laboratory Studies

The fundamental laboratory studies reported by the sub-committee were made to determine the polarization effects of alternating current on electrolytic cells. Using a 1/10 normal solution of sodium chloride for the electrolyte, tests were made where both electrodes were copper, both zinc, and also where one electrode was copper and one was zinc. The circuit was arranged so that various magnitudes of direct current and various magnitudes of alternating current, either separately or superimposed, would pass through the cell. Among the results found under this particular test set-up were: that zinc electrodes polarize less than copper electrodes; and that it required about 50 times as much alternating current as direct current to cause comparable polarization.

In its investigations the sub-committee has observed differences in the chemical composition of water supply; in the nature of the permissible impurities in the copper tubing installed; in the number and arrangement of galvanic couples resulting from the more or less

indiscriminate use of many different metals and variations in composition of the same metals; in the temperature of the hot-water supply; in the rate and amount of water drawn from the mains; and in many other factors, as well as the possible effect of stray current. It has not been possible to develop thoroughly all of the complex variables present in the different situations investigated, and the sub-committee's work has not progressed as rapidly as hoped, due to its inability to obtain a sufficient number of cases for investigation. The report of the sub-committee stated, however, that in none of the cases investigated was there evidence to indicate that the flow of alternating current over water pipes or mains, by itself, has caused damage to the piping or to the water flowing in the pipes; but the investigations have not progressed sufficiently to prove that such damage *could not* occur.

Future Program

Due to the complexity of the variables, the Grounding Committee has recommended continuation of the established program of investigating all complaints of contamination of water, where grounding of electrical circuits is involved, to determine if possible whether there is any relation in normal practice between superimposed alternating currents and effects on water pipes and the pipe contents. It also proposes to establish simple test set-ups where accelerated reactions might be obtained if such a relation does exist. These set-ups will have to be of such a nature as not to require continuous observation, but will be located in water plants or laboratories where occasional checks of the results can be made from time to time under the supervision of skilled personnel. It continues, however, to be important that all complaints should be investigated as promptly as possible, in order to expand the field experience and to establish correlation among the various factors involved.

ABSTRACTS OF WATER WORKS LITERATURE

Key. **31**: 481 (Mar. '39) indicates volume 31, page 481, issue dated March 1939. If the publication is paged by issues, **31**: 3: 481 (Mar. '39) indicates volume 31, number 3, page 481. Material enclosed in starred brackets, ★[ ]★, is comment or opinion of abstractor. Initials following an abstract indicate reproduction, by permission, from periodicals as follows: *B. H.—Bulletin of Hygiene* (*British*); *C. A.—Chemical Abstracts; P. H. E. A.—Public Health Engineering Abstracts; W. P. R.—Water Pollution Research* (*British*); *I. M.—Institute of Metals* (*British*).

HYDRAULICS AND HYDRAULIC ENGINEERING

Some New Aspects of Hydraulics for Water Works Engineers. George E. Barnes. Ohio Conf. Water Purification, 19th Ann. Rept. p. 93 ('39). Developments in field of hydraulics and fluid mechanics discussed. Attention drawn to fact that empirical formulas are sometimes limited to sharply defined range of application. Easy to forget circumstances under which formulas were originated and assumptions on which they are based. Demand for efficient aircraft has led to better understanding of mechanics of fluid flow. Application of new methods is illustrated by showing how to form a rational equation, how such methods have assisted in re-evaluating concept of friction factors in pipes, and how studies on hydraulic models can be utilized in determining probable performance of hydraulic structures. If certain specifications are followed, geometrically correct models can be made to produce perfect picture of what will happen on large scale.—*R. E. Thompson.*

Hydraulic Investigation of a Small Water Works System and Effects of Main Cleaning. Richard H. Ellis. J. N. E. W. W. A. **54:** 155 (June '40). Illustration given of methods used in hydraulic investigation of system in Millers Falls, Mass. (pop., approx. 1,800; ave. daily consumption, 44,500 g.p.d.). Water obtained from Lake Pleasant supplying both Millers Falls and Turners Falls systems; fire flow tests indicated deficiency in trunk main to Millers Falls consisting of 11,000′ of 10″ cast-iron pipe, only 600 g.p.m. being available at 20 lb. in business district. Pipe cleaning raised fire flows, raised W. & H. coefficient to 115 and 122 and reduced cost of pumping water 29%.—*Martin E. Flentje.*

Carrying Capacity Losses of Water Pipes in Service. Luis Valenzuela. Rev. de Explotacion de los Servicios de Agua Potable y Alcantarrillado de Chile. Nos. 16-17: 3 (May, July '39). Discrepancies found between per cent reduction in pipe carrying capac. given by Darcy's formula and W. & H. "trend curves" and results of flow measurements made in 14 pipe lines. ranging

from 165 to 684 mm. internal diam., which had been in use from 2 to 46 yr. in different localities in Chile, have convinced author of limited applicability of above methods for predicting pipe capacity losses after time of service. Better agreement between actual and predicted results by procedure suggested, in which use is made of Colebrook and Masey White formulas, based on theory of Von Karman-Prandtl, in regard to turbulent movement of a fluid, and on latest expts. made by Nikuradse with artificially roughened pipes, as well as on hypotheses that modification of hydraulic conditions in a pipe line is due entirely to increase in roughened surface and that size of irregularities which form roughened surface increase in direct proportion to time. Value of Chezy's coefficient "C" for new pipe as a function of time and roughness is given by formula:

$$C_{t_0} = 2\sqrt{8g} \log \frac{3.7D}{\alpha t_0} \qquad (1)$$

where D = pipe diam. in meters, α = roughness coef. in m. per yr., and t_0 = time in yr. necessary to acquire roughness equivalent to viscous action of water. For pipe which has been in service T yr., formula is:

$$C_{t_0} + T = 2\sqrt{8g} \log \frac{3.7D}{\alpha(t_0 + T)} \qquad (2)$$

Making

$$\frac{C_{t_0}}{2\sqrt{8g}} = p_0 \qquad (3)$$

and

$$\frac{C_{t_0} + T}{2\sqrt{8g}} = p \qquad (4)$$

in Equations (1) and (2), we have:

$$10^{p_0} = \frac{3.7D}{\alpha\, 10^{p_0}} \qquad (5)$$

$$10^{p} = \frac{3.7D}{\alpha\ t_0 + T} \qquad (6)$$

Combining Equations (5) and (6), and solving for α, we have:

$$\alpha = \frac{3.7D}{T} (10^{-p} - 10^{-p_0}) \qquad (7)$$

Also, from Equation (5):

$$t_0 = \frac{3.7D}{\alpha\, 10^{p_0}} \qquad (8)$$

Procedure for forecasting loss in pipe capacity after a no. of yr. of service is explained. From values for C_{t_0} and $C_{t_0} + \tau$ found in 2 actual flow measurements (one made on new pipe, and other after pipe has been in use for 2 or 3 yr.) values of p_0 and p are calculated by equations (3) and (4). With these values, coefficient of roughness a is obtained from equation (7) and t_0 from equation (8). With this data, value of Chezy's coefficient "C" for any time of service may be calculated by means of equation (2). Since carrying capacity of pipe is directly proportional to value of "C," per cent loss in capac. of pipe at any time will be given by per cent reduction in value of "C" for that time. Due to fact that formulas used are based on hypothesis that roughness coef. for given pipe remains constant at all points of "trend curve," and since it is an established fact that rate of pipe surface deterioration varies with characteristics of water carried, follows that forecast method described above will only apply to pipes which carry same type of water during their time in service. Making use of data presented in '35 Report of Committee on Pipe Line Friction Coefficients of N.E.W.W.A. (for systems which fulfilled condition stated above) and results of expts. made in Chile, author shows that if pipes are divided into 2 groups (one of pipes up to 200 mm. I. D. and other for larger diam.) per cent loss in capac. as estimated by new method corresponds more closely to actual determined losses than that obtained by previously suggested procedures. Another advantage of proposed method is that it permits forecast of future pipe capac. losses for any kind and size of pipe, and for particular characteristics of water which it is carrying, after 2 flow measurements have been made. In systems where no actual measurements are available, capac. losses may be predicted from data obtained, by suggested method, for a given size and kind of pipe in places where water of same physical and chemical characteristics is used.—*J. M. Sanchis.*

The Reduction of Carrying Capacity of Pipes with Age. Discussion of previous paper. J. Inst. C. E. (Br.) No. 8: 381 (Oct. '38). M. R. BARNETT: Quotes Brown's classification of deposits in pipes as: (1) incrustations on unprotected or imperfectly protected iron pipes; (2) deposits on inner surface of channels, nature of deposits depending on composition of water, and occurring over whole surface covered by water; and (3) accumulation of deposits in inverts. Author stated that 44" main suffered reduction of 13% capac. in 13 mo., due to combination of restriction of channel and increased friction. Amount so large necessitates rigid proof that some other factor was not working, such as a forgotten meter. Well-known fact that incrustation did not continue to grow indefinitely until it choked pipes of larger diam. had evidently been ignored. Another fact, apparently overlooked, is that there would be insufficient metal present to form tubercules and, at same time, withstand internal bursting pressure. Concluded that when all other causes of loss of capac. have been separated from total reduction due to "ageing" remainder might be set principally against formation of the black, slimy, viscous growth on interior surface of pipes, or against incrustation of pipes. In all probability rate of reduction of capac. would decrease after 20 yr. and would cease altogether in 100 yr. F. V. A. E. ENGEL: Criticizes some of refs. given by authors and asks further information concerning constants used in author's paper.

E. H. ESSEX: Observes that authors had gone over much ground which had been previously discussed by mathematicians with no new development. *Authors' Closure*: State that their paper had sound theoretical basis, supported by subsequent tests to check dimunition of flow which was so incredible to Mr. Barnett. Mr. Essex's proposed formulas for rough surfaces are widely at variance with facts and, in opening sentence, he fell into two errors of importance. Hence, authors feel that paper is a contribution to knowledge of subject and presents a formula of value.—*H. E. Babbitt.*

Flow of Liquid through Porous Media. ERIK G. W. LINDQUIST. Teknisk Tidskrift, Uppl. E. (Sweden) **70:** 49 (Apr. '40). Appreciation is expressed of great amount of research on rates of filtration now being carried on in U. S. For slow sand filtration, results of Hazen are still regarded as standard. For rapid sand filtration, work still continues. That of Hulbert and Feben is reported in the JOURNAL (Jour. A. W. W. A. **25:** 19 (1933)) under title "Hydraulics of Rapid Filter Sand." Formula at which they arrived was:

$$h = \frac{9.84}{1000} \frac{v_0 L}{d^{1.89}(T + 20.6)}$$

where h is loss of head in ft.; L, depth of sand in in.; v_0, rate of fitration, in m.g.d. per acre; d, diam. of sand grains in mm., and T, temp. in °F. When transformed to metric units, formula becomes:

$$h = 6060 \frac{v_0 L}{d^{1.89}(T + 29.22)}$$

or,

$$v_0 = \frac{1}{6060} d^{1.89}(T + 29.22) \frac{h}{L}$$

where h and L are now expressed in meters; v_0, in m. per hr.; and d, as before, in mm. From this formula it is obvious that Darcy's law, according to which rate of filtration varies directly with loss of head, is obeyed. In closely analogous case of flow through capillaries of uniform diam., rate varies directly with square of diam. In sand layer where grains are of uniform size, mean diam. of path of water must vary directly with diam. of sand. Hence it might be expected that filtration rate should vary directly with square of diam. of sand rather than with lower power. No term in formula which represents porosity, while influence of viscosity is expressed indirectly by means of temp. Therefore, author was led to enquire whether it might not be possible to find a formula which would: (1) be dimensionally correct, and (2) give to porosity and viscosity their due weight. He succeeded in attempt and proved accuracy of formula from Hulbert and Feben data. Started by treating filter as though it were a bundle of similar capillary tubes ranged side by side; these capillaries are constricted in diam. by sand grains protruding into them, but, as sand is of uniform size, one capillary was assumed to be very much the same as another, and mean velocity, v_1, of the water passing through to be

practically same in each. If, then, a is cross-sectional area of each capillary, and A its internal area, product ha, representing total disappearance of energy in the capillary, must be equal to resistance offered by capillary to flow and therefore proportional to product, Av_1^2, of total internal area of capillary into square of mean velocity of flow. Proceeding from this assumption by a series of steps too detailed for abstraction, author, who is a resourceful mathematician, arrived at dimensionally accurate formula:

$$v_0 = \frac{g}{36K} \frac{\psi_1}{(1-\psi)^2} \frac{d^2}{\nu} J$$

where K is a constant; g has its usual signification; ψ is the porosity; ψ_1 is the ratio of v_0 to v_1; ν is the kinematic viscosity coefficient $\left(\text{the denominator of the Reynolds number, } \frac{v_1}{\nu}\right)$ expressed in sq. m. per sec.; and J is equal to $\frac{h}{L}$. The relation between ν and T, the temp. (C.), is given by Poiseuille as:

$$\nu = \frac{1.775}{1 + 0.03368T + 0.000221T^2} . 10^{-6}$$

Rate of filtration varies as square of diam. of sand; porosity (in ψ and ψ_1) and viscosity are given due weight. It remains to test out the formula on the Hulbert and Feben data. Eight different sizes of sand were used. Following particulars of each are tabulated: (1) limiting Tyler sieve sizes used for grading; (2) size expressed in mm. mesh width; (3) ave. diam. of sand as determined by weighing, under microscope, and as geometric mean of limiting sieve mesh dimensions—close agreement between 3 methods being noticeable; (4) uniformity coefficient (Hazen); and (5) porosity under experimental conditions. Tests were carried out in vertical cylindrical glass tubes, 6" in diam., and in which sands were allowed to settle under uniform conditions. Filtration rates of from 0.9 to 10.8 m. per hr. were employed and sand depths of from 0.25 to 0.75 m. Ample confirmation was obtained of the law that when rate of filtration and temp. remain constant, loss of head varies directly with depth of sand bed. Data which Lindquist used to test out his formula are listed in another table wherein appear results of 40 tests, 5 on each of 8 different sands. In all of these both rate of filtration and depth of sand bed were constant, former at 4.871 m. per hr. and latter at 0.254 m. Temp. was varied over whole range from about 2°C. to about 21°C. and corresponding loss of head in each case was observed. Data in this second table include Hulbert and Feben figures for (1) sand serial number; (2) loss of head in ft.; (3) temp. F.; which are supplemented by Lindquist to include (4) loss of head in m., i.e. h in formula; (5) temp. C.; (6) viscosity coefficient $\times 10^6$ (ν); and (7) value of $\frac{h}{\nu}$. Lindquist also inserts, for each sand separately, ave. value of $\frac{h}{\nu}$. It is immediately obvious that to each different sand size corresponds a virtually constant value of $\frac{h}{\nu}$. This is important, because it is strong evidence for validity of

Lindquist formula, for, if in that formula we substitute for J its value, $\frac{h}{L}$, it becomes obvious that $\frac{h}{\nu}$ is a factor on the right hand side, and that for any one sand under given experimental conditions, everything else is constant, and therefore $\frac{h}{\nu}$ must be constant. In order finally to verify formula on the H. and F. data, 2 small difficulties have to be cleared up. First, ψ_1, has not been determined—it must therefore be assumed that it is proportional to ψ, and write ψ for ψ_1, allowing the constant to take care of the proportionality factor. The error involved must be negligible. Other difficulty arises through fact that in deriving the formula, an expression was needed for total wetted area in filter, which area includes not only surface of sand, but also

TABLE 1

Test of Constancy of K_1

SAND SERIAL NUMBER	SAND DIAMETER	CALCULATED VALUE OF K_1 WHEN CORRECTION β IS:	
		Omitted	Applied
	mm.		
1	0.289	0.3212	0.3196
2	0.374	0.3381	0.3363
3	0.435	0.3340	0.3320
4	0.510	0.3436	0.3412
5	0.614	0.3405	0.3375
6	0.709	0.3362	0.3331
7	0.899	0.3387	0.3345
8	0.977	0.3370	0.3330
Mean		0.3366	0.3334

that of filter walls in contact with sand, which latter is usually altogether negligible, and was regarded as such. But in filters so small as 6″ in diam., some correction is necessary, and Lindquist shows how to work it out in form of a divisor, β, to be applied to d in the formula. In a third table are assembled following data; (1) sand serial number; (2) sand diam. in mm.; (3) porosity; (4) value of correction β; and (5) value of $\frac{\psi}{(1-\psi)^2}$. Applying adjustments referred to and collecting together all constants common to H. and F. data into one new constant, K_1, formula may now be written as:

$$K_1 = \frac{\psi}{(1-\psi)^2}\cdot\frac{h}{\nu}\cdot\left(\frac{d}{\beta}\right)^2$$

after which constancy of K_1 must be tested. This has accordingly been done

and results are given in Table 1, which is crucial test of validity of formula. In this, deviation from mean is seen to be inconsiderable, and no more than might be due to unavoidable errors of observation when it is remembered that determinations of loss of head and rate of filtration were made with an accuracy of 1.5 mm., or 2%. In conclusion, it is pointed out that when sieve analysis of filter sand is known, then, owing to hydraulic grading which takes place in back-washing and renders filter into a series of superimposed layers, each of which is approximately uniform in diam., the actual effective size, d_e, of sand in question; that is to say, the diameter of that uniform sand which when used to replace given sand would give same results, is given by the formula:

$$d_e = \frac{100}{\frac{b_1}{d_1} + \frac{b_2}{d_2} + \frac{b_3}{d_3} + \cdots}$$

where b_1, b_2, etc. are percentages by weight of different sieve fractions, and d_1, d_2, etc., the corresponding diameters, it being assumed that all grains are approximately spherical.—*Frank Hannan.*

Construction of Manometers for Measuring Flow. A. D. LORING. Ind. Eng. Chem.-Anal. Ed. **11:** 626 (Nov. '39). One instrument described is built on U-tube principle but one arm is reservoir of large cross section and for practical purposes, only level in tube of small cross-section need be read. Also shown is device for measuring flow of cooling water, using water itself as manometer liquid in gage which is essentially an inverted equal-bore U-tube with pump to regulate air pressure above manometer water columns and thus adjust lower level column to a scale zero point. Air pump is also operated by water whose flow is being determined.—*Selma Gottlieb.*

Discharge of V-Notch Weirs at Low Heads. FRED W. BLAISDELL. Civ. Eng. **9:** 495 (Aug. '39). General equation for discharge over V-notch weirs is:

$$Q = KH^{5/2} = C_d \frac{8}{15} \tan \frac{\alpha}{2} \sqrt{2g}\, H^{5/2}$$

in which α is interior angle of notch, H is depth of flow, and C_d is weir constant. Tests seem to indicate that V-notch weir is not dependable measuring instrument at heads below 0.3'.—*H. E. Babbitt.*

Free Discharge of Fluids Through Small Circular Orifices. HIRA LAL ROY AND NIRMAL K. SEN-GUPTA. Ind. Eng. Chem. **32:** 288 (Feb. '40). According to simplified equation derived from Bernoulli's theorem, rate of discharge through the same cylinder and the same orifice under the same head is the same and is independent of the nature of the liquid. Authors undertook to test this equation using head of 2' to 6', orifice diam. of $\frac{1}{16}$" to $\frac{3}{8}$" and 5 liquids (including water) with widely varying specific gravities, surface tensions and viscosities. For water, C, coefficient of velocity for orifice,

decreases with increasing head for all orifices except the $\frac{1}{16}''$, probably due to imperfect contraction at low head and for orifice diam. under $2\frac{1}{2}''$. Apparently for every liquid with a certain viscosity and surface tension, a critical diam. and critical head exist below which C will decrease with decreasing head and diam., and above which it will increase. Probable that for all liquids C is constant above a certain critical diam. and head. Possible, too, that there is a relation between critical velocity and orifice diam. for any particular liquid, so that below point of critical velocity, C will decrease with decreasing diam.; reverse being true above diam. where critical velocity is exceeded. This is apparent for water where C increases continually with decreasing orifice diam., whereas in other liquids it is decreasing. Graph illustrates probability of existence of such a tendency.—*Selma Gottlieb.*

Error in Discharge Measurement Due to Transverse Slope of Weir Crest. WARREN E. WILSON. Civ. Eng. **9:** 429 (Jul. '39). Horton's formula is

$$Q = \frac{2CL}{5(H_2 - H_1)} (H_2^{5/2} - H_1^{5/2})$$

in which Q is rate of discharge, C is weir coefficient, H_1 is head at one end of weir and H_2 is head at other end, and L is length of crest. Percentage error is very nearly

$$e = \frac{100\, s^2 L^2}{32H^2}$$

in which $s = \dfrac{H_2 - H_1.}{L}$ For a triangular weir percentage error is 100 $\tan^2 \alpha$, where α is angle between the bisector of notch angle and a vertical line.—*H. E. Babbitt.*

Pressure-Momentum Theory Applied to the Broad Crested Weir. H. A. BOERINGSFELD AND C. L. BARKER. Proc. A.S.C.E. **65:** 1719 (Dec. '39). Broad crested weir is advantageous as a measuring device where head is at a premium. Expression for flow over weir can be written:

$$Q = \Omega b H^{3/2}$$

in which

$$\Omega = \frac{\sqrt{2g}}{2} \left\{ \left(\frac{K^2 - 1}{K^2} \right) \left[\frac{d_4 + H}{K(d_4 + H) - H} \right] \right\}^{1/2}$$

and Q = rate of flow, b = breadth of weir, H = head of water above the top of the weir, d_4 = height of weir above bottom of channel, g = acceleration of gravity, and K is a constant fixed by the weir. Has been found that formula is insensitive to the values of K and that value of 2 can usually be assumed. Accuracy of formulas is to be questioned if breadth of weir crest is so small that water surface does not become parallel to surface of weir, or nearly so.

Discussion. Ibid. **66:** 563 (Mar. '40). J. C. STEVENS: As a metering device broad-crested weir may have advantages under some circumstances. Rounding of upstream curve will probably have effect of increasing discharge. Deposit of sand, silt, or sludge against upstream face will also increase discharge, since it lessens upstream reaction to dynamic force against front face. Parshall flume has decided advantages over this type of weir because it keeps itself generally free from sediment and calibrations cover sizes from 3" to 50' crest widths and include both free-flow and submerged-flow conditions. There is need for open-channel metering device which is indexed by far smaller heads than is possible with any type of free-flow weir. Writer believes that submerged Parshall flume and submerged orifice are best devices so far developed to meet such conditions, largely because of exhaustive calibrations they have received. H. G. WILM: No successful attempt yet been made to develop device for water measurement that could be "rated" on purely theoretical basis. One assumption which, perhaps, results in most deviations of theory from practice, is that water is without viscosity and hence without friction. Another is that acceleration, deceleration, and changes in direction of water flow are accomplished without expenditure either of time or space. Throughout paper, apparently, assumption is made that parallel flow will occur over level weir surface, except where flow is influenced by end conditions. These arguments are by no means intended to indicate that broad-crested weir is unsuited to measurement of open-channel discharge. Furthermore, when properly designed and constructed, broad-crested weirs can be calibrated accurately on a rational basis. In order to do so, however, several requirements must be met in design. Writer, with others, worked on development of flume for use in measuring open-channel flows containing bed loads. Functions on same principle as broad-crested weir. Essential difference in design involves only use of horizontal transitions instead of bottom contraction to accomplish development of critical flow. Preliminary results of these studies published in '38. Principles involved and results of studies should be applicable to any broad-crested weir that conforms in design and construction to the requirements. *Ibid.* **66:** 1119 (June '40). JOHN W. HACKNEY: In an equation, authors define relationship between weir coefficient (Ω), head on weir (H), and height of weir face (d_4). If, as authors suggest K may be assumed to be a constant and equal to 2.00, this formula may be reduced in the following manner:

$$\Omega = \left[12.1\left(\frac{1 + H/d_4}{2 + H/d_4}\right)\right]^{0.5}$$

The curve representing the equation is a good mean curve for the test points. It is not true, however, (in a comparison with tests made by Bazin, Woodburn, and Horton in which a mean value for Ω is about a constant at 2.6) with an unexplained widening of the band of test points below $H/d_4 = 0.3$. THOMAS H. PRENTICE: Authors direct particular attention to necessity of obtaining, accurately, least flow depth over weir, since it is at that depth that parallel flow is assumed to exist. Another assumption of pressure-momentum theory is that pressure distribution on upstream contracted section is hydrostatic.

Manometer readings of the piezometer connections on vertical upstream face of weirs showed that pressures were substantially hydrostatic for lower points on face. There should be considerable caution in applying formula in cases where there is curvilinear flow over entire breadth of the crest. Boris A. Bakhmeteff: Author's findings center around an equation which constitutes a "discharge coefficient," the numerical value of which can be determined by inserting into the formula two easily measurable quantities. Questionable whether very thought of using entrance section of broad-crested weir as a metering device is expedient, particularly since Rouse disclosed the adsorption of the exit section as a possible control. D. D. Curtis: Evident that authors ignored certain items in addition to mentioned friction loss between crest and section of parallel flow. To afford comparison with authors' results, data were recomputed for runs made by the writer. Fact that value of K obtained under decidedly different conditions closely approached that found by authors gives weight to belief that items disregarded are unimportant in their effect. In any event, size of Ω is not particularly sensitive to value of K. Calculations show that reducing K from 2 to 1.97 would make a difference of $<1\%$ in Ω. Therefore, use of value 2 appears warranted. Carl Rohwer: Broad-crested weir cannot take place of sharp-crested weir in lab., and in field it has some of same limitations that restrict use of sharp-crested weir. Principal objection is that pool formed in front of weir is natural settling basin. Accumulation of material in pool reduces height of weir and also increases velocity of approach; consequently formulas generally applied give erroneous results. Control sections, such as the Parshall flume, are more satisfactory than weirs for use in streams and canals. John Hedberg: On a slope, steep enough to warrant use of cosine of angle of inclination, pressure is definitely not equal to weight of column of liquid above it. More exact value is considerably less than this. Another point that deserves attention is role of hanging eddy along upstream edge. It acts like an additional height of weir. Possibly, fluctuations in size of this vortex space are major factors in the variations in value of K.—*H. E. Babbitt.*

Flash-Board Pins. *Closure of discussion.* Chilton A. Wright and Clifford A. Betts. Proc. A.S.C.E. **66:** 1213 (June '40). Formulas developed in paper will not apply to pipes set horizontally in the abutments. Use of hinged flash boards saves the boards (preventing them from floating away at high water). Characteristics of standard pipe make it structurally and economically adapted to use for flash-board support. Reduction of pressure at top of flash boards, due to overflow, is such a minor factor that it can be ignored except in unusual cases of very high approach velocities. As a practical matter, degree of vacuum behind flash boards would depend upon height of flash boards, clearance at end of crest, width of flat channel immediately below boards, and other factors; and should receive consideration in design of an installation.—*H. E. Babbitt.*

Relation of the Statistical Theory of Turbulence to Hydraulics. A. A. Kalinske. Proc. A.S.C.E. **65:** 1387 (Oct. '39). A fluid at any point is in a state of turbulence if direction and magnitude of its velocity vary with time.

It manifests itself in the form of large and small eddies, whirling about from one layer of fluid to another. Complete and mathematically rigorous theory of turbulence has not yet been developed. It is a basic principle that transverse velocity fluctuations follow normal probability distribution. If turbulence is uniform in direction of flow, energy present due to turbulence need not be taken into account in any energy equation. However, if energy of turbulence varies from point to point, such as occurs in conduit expansion, beyond bends, etc., its omission, in any study of energy changes, results in error. Energy of turbulence is dissipated into heat, due to viscosity, and cannot be recovered. Rate at which potential energy is transferred into turbulent energy is not, necessarily, equal to rate of energy dissipation into heat, at any one point. Turbulent energy can be diffused to other points by action of eddies and then dissipated into heat. Turbulence eddies are capable of causing exchange of momentum, heat, matter, etc., from one layer of fluid to another. Knowledge of diffusing power of turbulence is most important in studies of suspended material transportation. Experimental methods and equipment used to determine various quantities involved in turbulence include: hot-wire anemometer in wind tunnels and in water at low velocities; photographing on moving picture film illuminated or colored suspended globules of carbon tetrachloride and benzine, having a specific gravity equal to that of water, but immiscible with it; and photographing on motion picture film a thin color stream injected into flowing water through a fine needle. Cup type of current meter is sensitive to turbulence but does not follow velocity fluctuations exactly because of its inertia. In a conduit of expanding cross section, additional losses of energy are directly attributable to creation of turbulence energy beyond that ordinarily present in turbulent flow. Rate at which potential energy per unit length of circular conduit is converted into turbulent energy is:

$$Q \frac{h_f}{L} = 2\pi \int_0^r \rho \frac{du}{dy} y \, dy$$

in which $\frac{h_f}{l}$ equals loss of heat per foot of conduit; Q equals total weight of water flowing in unit time; ρ equals unit shear stress at y distance from center, r is pipe radius; and U is mean velocity parallel to pipe axis. In problems of silt sedimentation, diffusion characteristics of turbulence are important. Two general problems to illustrate this are analyzed. Principles of turbulence may explain why surface velocity in open-channel flow is less than maximum, entrainment of air by a high surface velocity, and relation of turbulence to accuracy of Allen salt-velocity method of measuring discharge. Instruments for measuring mean velocity and static pressure in a turbulent stream, with intensities of turbulence ordinarily present, are but slightly affected by turbulence, anticipated errors due to turbulence being less than 1%. *Discussion. Ibid.* **66**: 165 (Jan. '40) HUNTER ROUSE: Eddies produced by local disturbances are by no means synonymous with true turbulence. Has been suggested that new word be devised to distinguish large-scale "turbulence" in rivers from small-scale turbulence in pipes, although former really consists of local eddies

bearing same relation in river as that of small eddies in pipe. Many so-called energy dissipators could be made more effective and less dangerous if it were realized that formation of large-scale eddies effectively reduces mean velocity of flow, and that presence of such eddies is sometimes as harmful as that of high mean velocity. Of primary significance is fact that both length scale and velocity scale of turbulence govern processes of diffusion and energy dissipation. *Ibid.* **66**: 352 (Feb. '40) Martin A. Mason: Among those familiar with modern thought on subject, turbulence is generic term modified as isotropic, non-isotropic, or anisotropic, initial, fully-developed, etc., each designation indicating to experienced reader a characteristic phenomenon. To practical man who is not familiar with math. treatment of turbulence they are meaningless. Thus for purpose of studying theory of turbulent flow may be convenient to define turbulence as being of an entirely random nature without definite periodicity of time. With this definition it is to be noted that present statistical theory of turbulence does not offer immediate possibility of application to most hydraulic problems as it is chiefly concerned with isotropic turbulence. From hydraulicians point of view determination of time period necessary for required dissipation of energy in form of anisotropic turbulence is perhaps more important than determination of rate of dissipation. Should be remembered that similar Reynolds numbers for model and prototype do not alone guarantee similar conditions of turbulence unless there is complete geometric similarity. Possibility of existence of scale effect should justify attention of model experimenters. In any photographic method employing foreign particles to reveal motion of water some uncertainty attaches to assumption that they follow exact motion of water particles. Photographic method, involving motion pictures of entrained aluminum particles, utilizing stroboscopic light, has been developed in France. Arrangement permits determination of velocities within about 1% error, and determination of position in space with about 5% error. J. C. Stevens: Lately hydraulic engineers have recognized that energy gradient in tortuous conduits is quite uniform and that energy losses are not concentrated at bends as was long believed. So-called losses are merely heat dissipation through viscous properties of water in turbulent motion that continues downstream far beyond origin of turbulence. There is need of means and method of measuring velocities at any point in flowing stream without inserting something that itself causes turbulence. Perhaps more simple means than that of motion picture camera will be found. *Ibid.* **66**: 709 (April '40) Clyde W. Hubbard: An inference to be drawn from a statement in author's conclusion is that coefficient of pitot tube is larger than unity. This is contrary to experience. Author's statement that error of well-designed pitot tube should be $< 1\%$ is not in accordance with writer's experience of ordinary errors of 2% with as much as 5% caused by turbulence. Possible that when scale (mean eddy size) as well as intensity of turbulence are both measured successfully, some rational explanation of error of pitot-static orifice will be forthcoming. Errors caused by turbulence can be avoided by use of a type of tube that is but slightly affected and by calibrating it under flow conditions similar to those under which it is to be used. John S. McNown: One apparent difficulty with photographic measurements is indicated by angular character of curve showing relationship between time and

instantaneous velocity fluctuations. Increase in number of pictures taken per sec. should smooth out curve, but would increase difficulty in analyzing individual frames. Author does not give conditions under which data used for analysis of λ were obtained nor degree to which isotropy assumed in development was attained. Effect of turbulence should be investigated in standardizing results of comparable research problems. Difficulty is probable absence of isotropy in turbulence near boundary where its effect is most important in this problem. SAMUEL SHULITS: From about middle of 19th century to present there may be said, without rigid accuracy, to be 3 phases in development of hydraulics and fluid mechanics. First was a prolific stage of empiricism. Second, use of analogies and imaginative concepts. Present is beginning of third phase. Recent statistical theory of turbulence is more accurately embryonic theories and facts of turbulence examined in light of laws of chance and statistics. It is not new theory—only dawn of orderly research. *Ibid*: **66:** 581 (Mar. '40) BORIS A. BAKHMETEFF: Drawback of all cinematographic methods of measuring turbulence is their natural tediousness. So far most tangible contributions of statistical approach are not in realm of general and all-embracing "theory." Statistical treatment has matured into certain well-established basic definitions, which permit one to appraise and characterize turbulent phenomena in plausible numerical terms. Foregoing remarks tend to show how uncertain, as yet, is current knowledge of inner mechanism of turbulence. *Ibid.* **66:** 967. PAUL NEMENYI: Author distinguishes 2 main practical problems: (1) disposition of energy dissipators all along a conduit in order to convert potential energy into turbulent energy, rather than into kinetic energy of main longitudinal flow; and (2) energy dissipation at end of a chute to convert kinetic energy of main flow into more dispersed form of turbulent energy and into heat. Writer has been interested in first problem, which is important in the design of fishpasses, timber-floating channels, and sometimes pipes of surge tanks, and is related also to spillway design. With little exception statistical inquiries into turbulent flow have been concerned with cases in which average flow is steady, straight, and uniform. Obviously no rough conduit can strictly satisfy these conditions; and as size and efficiency of "roughness" increases, local deviations of average flow from straight line will also increase in importance. This is one reason for inadequacy of present-day turbulence statistics in producing satisfactory treatment of energy dissipators. Results of intense momentum transfer produced by energy dissipators discussed can well be compared to a shock or impact, in contrast to an abruptly expanding conduit, for which expression "shock loss" is quite unsuited. When considering problem of destroying large forward velocities, it should be kept in mind that task is not always best solved by dissipating energy at a particularly high rate. May be more advisable to dissipate energy at somewhat more moderate rate, if it can be located conveniently. Writer believes that most important field for civil eng. applications of turbulence statistics lies in problem of sedimentation transportation in canals and rivers. Second half of experimental problem consists of how to record movement of particles made visible. A uniformly moving camera might be used with speeds of systematically varied magnitudes. BENNIE N. NETZER: For quick practical results some method that is less tedious

and more flexible than the photographic is desirable for a thorough understanding of turbulence mechanism. An instrument that is very sensitive to turbulence and will measure the instantaneous max., mean, and min. velocities at any point has been designed. Instrument is a helical spring dynanometer that is calibrated by measuring velocity of a jet of water moving with uniform velocity, and then determining elongation of helical spring in same jet. *Closure of discussion*: *Ibid.* **66**: 1229 (June '40): Boundary regions in a straight conduit are major source of turbulence. Physical law controlling creation and shedding of eddies from viscous boundary layer may some day be formulated. True turbulence is probably best defined as being motion in which there is no periodicity in magnitude of three velocity components (normal to each other). Turbulence must be considered in study of sediment transportation as bed load. One of most urgently needed fundamental experimental investigations in regard to sediment transport is study of zone near bed, where bed material is picked up and transported in suspension. In addition to effect of turbulence, an overestimation of discharge results from pitot-tube readings. This should not ordinarily exceed 0.5%, however, except when a large tube is used in a small pipe. It is not a law of statistics that deviations from the mean of any large number of measurements will be distributed according to normal error law. Statistical theory of turbulence does not in any way obviate necessity of observations and measurements; however, it does give direction to such research work. Ordinary turbulent flow in conduits is result of dispersion of eddies from boundaries into main body of a fluid stream. Ordinary roughness projections are not effective for producing a large retardation of forward velocity. Photographic methods of measuring fluctuating velocities have many advantages, even if they are time-consuming, since no apparatus is introduced into stream. Excellent data on velocity fluctuations in large streams can be obtained with current meters. Before suspended material can be determined accurately and economically, it is important to know how the concentration fluctuates; this fluctuation in sediment concentration is closely allied with velocity fluctuations of a turbulent stream.—*H. E. Babbitt.*

The Effects of Turbulence and Cavitation Upon Erosion and Corrosion. C. J. Odend'hal. J. Amer. Soc. Naval Eng. **50**: 2: 231 ('38). Theory is presented and discussed to account for 2 attack mechanisms: (1) effect of high water velocity and of oxidation due to entrained air present in liquid flow, and (2) deleterious effect when dissimilar metals form container for liquid flow, as they affect salt-water lines, condenser circulating systems, and boiler feed systems. Effect of dissimilar metals such as copper piping, bronze impellers, cast-iron pump bodies, and brass flanges in system is considered, and it is stated that all tube sheets, tubes, and heads should be of same material, and where heads are of different material they and bolts securing them should be insulated.—*I. M.*

Artesian Well Hydraulics by Unit-Head-Loss Method. M. A. Churchill. Civ. Eng. **10**: 307 (May '40). Derivation of formula usually employed in solving problems in artesian-well hydraulics is quite satisfactory from purely

mathematical viewpoint but fails to convey mental picture of physical processes involved. For purposes of this discussion will be assumed that aquifer, within area affected by well, is of uniform texture and of constant thickness. Must also be assumed that water in water-bearing formation is not moving before pumping of well begins, and that well completely penetrates aquifer. Under such conditions, water will flow toward well equally from all directions when pumping is in progress, velocity of moving water varying inversely as distance from center of well. Assume that aquifer, about well, is divided up into concentric vertical cylinders, each cylinder having a wall thickness equal to radius of well. If velocities of approaching water are plotted as ordinates, and distances from center of well, in radii of well, as abscissas, true ave. velocity of water, as it passes through walls of any cylinder, can be determined by measuring area under corresponding portion of velocity curve and then dividing this area by its width. Head losses occurring in walls of imaginary cylinders are in direct proportion to ave. velocities therein. Total drawdown in well must equal sum of losses occurring in cylinder walls between well and outer limit of circle of influence, assuming for the moment that friction loss and velocity head inside well bore are negligible. Plotted relation of head loss in aquifer to horizontal distance from well constitutes what writer calls unit-head-loss curve. Curve has general shape of drawdown curve of every artesian well drawing its supply from porous, non-fissured rock. Many practical problems involving relative discharge of well under various drawdown conditions and relative discharges of wells of different diam., can readily be solved by use of unit-head-loss method.—*H. E. Babbitt.*

The Calculation of Equilibria in Dilute Water Solutions. D. S. McKinney. Proc. A.S.T.M. **39:** 1191 ('39). Descriptions are given on method of calculation of equilibrium concentrations of constituents of water solutions, on law of conservation of matter, and on principle of electrical neutrality. Formulas are given relating equilibrium constants to other thermodynamic quantities such as free energy, heats of reaction, entropies of reaction and e.m.f. of cells. Examples selected from field of corrosion and of water treatment are used for illustration of use of various formulas.—*T. E. Larson.*

Notes on Hydraulic Jump. Fred C. Scobey. Civ. Eng. **9:** 467 (Aug. '39). Hydraulic jump may not always occur where it is expected. Jump has many uses such as: to dissipate energy; to recover head; to mix chemicals; for aeration; to reduce uplift; to increase discharge of an undershot orifice; and as an indicator of flow types. Jump can usually be expected after flow over a dam crest followed by a steep incline; after any channel flow above critical velocity; after flow under a gate shutter; or after a vertical fall with flow direction turned to horizontal. It may occur unexpectedly in: a sinuous canal of uniform shape with velocities just under critical; where excess fall is provided in any part of a variable channel; where constriction is designed for increase in velocity from a feeder canal but flow is less than design flow; in flumes with steep incline at upper end; in pipe flumes which may suddenly "plug"; and in a lined tunnel. It may fail to develop where discharge is into a pool and incline continues without change until deeply submerged, or where incline turns to-

ward horizontal below tailwater level. Law of conservation of momentum permits a satisfactory analysis of "standard," clean-cut jumps.—*H. E. Babbitt.*

Methods of Representing Distribution of Particle Size. J. B. Austin. Ind. Eng. Chem.-Anal. Ed. **11:** 334 (June '39). Distribution of particle size (PS) is commonly represented by plotting PS against number of particles having size shown (frequently distribution curve) or PS against per cent of total no. having diam. greater or less than size indicated (cumulative curve). In both cases relatively large number of experimental points is needed to fix position of curve, interpolation and extrapolation are unsatisfactory, and data cannot readily be converted from one form to other. Frequency distribution curve resembles probability curve and cumulative curve is S-shaped, resembling ogive or integrated probability curve. By using special coordinates, curves can be reduced to straight line, allowing plotting from meager data. Interpolation is easy and extrapolation reasonably certain. Most successful method is plotting PS on log scale and cumulative per cent oversize or undersize on probability scale; this gives satisfactory accuracy for pulverized silica, crushed quartz, clay, powdered alumina, etc. Graph paper with these coordinates is available. For few substances for which distribution of size approaches normal probability distribution fairly closely (e.g., zinc oxide produced by chem. process, not reduced by crushing or grinding) straight lines result when PS is plotted on linear scale and cumulative per cent oversize or undersize on probability scale. Third method, notably successful with broken coal, is plotting size of lump on log scale and cumulative per cent oversize on log-log scale. Method has been suggested for other brittle materials containing cracks at which cleavage may start. No one method applicable to all types of material. When log-probability plot is used, data from weight (screen analysis) curve can be converted to size basis as follows: $\log M_g = \log M_g' - 6.9078 \log^2 \sigma_g'$; where M_g is geometric mean of PS values, M_g' is geometric mean of PS values obtained from screen analysis curve, and σ_g' is geometric standard deviation derived from screen analysis curve.—*Selma Gottlieb.*

Treatment of Liquids Containing Suspended Particles With Sound and Ultra-Sound Waves. E. Hiedemann and O. Brandt. J. Soc. Chem. Ind. (Br.) **58:** 508 ('39). Turbid melts and solutions and other liquids are clarified by subjection in an elongated apparatus or in a continuous stream flowing through an elongated chamber, to waves of a frequency of 8-100 kHz. transmitted in long direction of liquid. Length of chamber should be multiple of its cross-section, and should preferably be tuned to resonance with impressed waves. Draw-offs for clarified liquid and sludge should be situated alternately at intervals of $\frac{1}{4}$ wavelength from source of waves. Graph given for finding best wave length according to size of particles of hydrosol.—*W. P. R.*

Putting a Damper on Surge. Paul Hansen and Paul E. Loangdon. Eng. News-Rec. **124:** 523 (Apr. 11, '40). In new Lake Erie water supply system for Toledo, 3 sections of conduit are subject to surge difficulties: (1) 108",

15,490′ conduit from intake to shore pumping station; (2) 78″, 47,500′ steel pipe line from shore station to filter plant and high-service station; and (3) 47,000′ connection from filter plant to distr. system, varying in diam. from 72 to 42″. In *1*, max. velocity is 4.25 ft. per sec. and surge problem is not critical, both ends of line being open. *2* is designed for max. pressure of 150′ and is subject to operating pressure between 25′ static and max. of 100′. Critical surge condition would exist if pumps delivering at max. rate of 120 m.g.d. lost power simultaneously. Without surge control devices, sufficient vacuum to collapse pipe line and pressure as high as 190′ might occur. Extended investigations indicated that best method would be to regulate min. pressure by admission of air and max. pressure by controlling closing cycle of pump discharge check valves. Two 24″ vacuum-type angle valves with 16″ orifices will be installed in line and each pumping unit is equipped with cone check valve. Ample factors of safety have been allowed and wide range of adjustments provided for to allow alterations to meet actual conditions. Principal danger in *3* is low or negative pressure near pumping station in event of loss of power to all pumps. In this section, a 1-mil. gal. wash water tank has been utilized as surge tank. Tank will also serve for storage of wash water and reserve storage in case of power failures of short duration. Tank and trunk main are interconnected through 2 surge relief valves designed to open fully in max. of 2 sec. when pressure on trunk main side is 4′ less than pressure on tank side. Station is served from 3 power sources and outages should be infrequent and of short duration.—*R. E. Thompson.*

A New Pipe Formula. LELAND S. RHODES. Eng. News-Rec. **125:** 213 (Aug. 15, '40). Shown that usual pipe-friction formula for loss of head expressed as function of Reynold's no. somewhat in error because does not take into account Froude's no. Following formula is developed:

$$h = 0.212 \frac{L}{D} \frac{V^2}{2g} \frac{N_F^{0.04}}{N_R^{0.19}}$$

in which h = loss of head, L = length, D = diam., V = velocity, g = acceleration due to gravity, N_F = Froude's no., and N_R = Reynold's no.—*R. E. Thompson.*

INDUSTRIAL WATER SUPPLY

Industrial Waters in Canada. *Interim Report No. 4.* HARALD A. LEVERIN. Bur. of Mines (Canada). Memorandum Series, No. 72. (Sept. '39). Fourth of series of reports of analyses of industrial waters in Canada. Previous reports give tabulated analyses of natural and treated waters, discussion general character of waters, analytical methods employed, various diagrams, and requirements and effect of water character on various Canadian industries. Deals with quality of surface and civic water supplies in western Canada, including British Columbia and other parts of area. British Columbia civic waters are on the whole very good, low in color and can be distributed, in most cases, to consumers without treatment. 120 tabulated complete chem. analyses given.—*Martin E. Flentje.*

Industrial Waters in Canada. *Interim report No. 5.* HARALD A. LEVERIN. Bur. of Mines (Canada). Memorandum Series, No. 77. (Jun. '40). Investigation of quality of Canadian waters used or available for industry and for civic supply was continued. Report deals with surface and civic supplies in northern mining and industrial areas in Ontario and Quebec, other places in Quebec, and in Maritime provinces. Analyses of samples from 30 key stations on larger lakes and rivers of industrial importance and from 73 civic supplies are tabulated and discussed briefly.—*R. E. Thompson.*

Requirements of Good Water Supplies for the Dairy Industries. K. G. WECKEL. Milk Dealer **29**: 7: 40 ('40). Water supplies used in dairy plants should be bacteriologically acceptable, free from particles, suspension, sediment and oil film, inert chemically, available in considerable volume, low in temp., and uniform in composition. Variability in chem. composition of water in 100 American cities is discussed in relation to its use by dairy industry. —*C. A.*

The Problem of Water in Paper Mills. F. STERPIN. Rev. Univ. Min. (Fr.) **15**: 549 ('39) (*Épuration des Eaux I*). Describes effect of sand, mud, iron, and algae in water used in manufacture of paper. Short description of composition of size, effect of water on each of its constituents, and action of size in water. In sizing of paper, aluminum sulfate is used to precipitate resins, which, in presence of aluminum hydroxide, adhere to cellulose fibers. A pH value of 5-6 is required for satisfactory precipitation. If calcium and magnesium salts are present in water, part of resin is precipitated in absence of aluminum sulfate and does not adhere to fibers during further treatment. Effects of protective colloids, such as gelatin and casein, and of excess aluminum sulfate in improving sizing process with hard waters are discussed. In reply to discussion, author stated that it had been proposed to use sodium bisulfate in place of amount of aluminum required to react with bicarbonates; this, however, had disadvantage that slight excess of sodium bisulfate would reduce pH of solution.—*W. P. R.*

The Correction, by Means of Ion-Exchange Materials, of Water Used in Dyeing. RICHTER. Meilliand Textilber (Ger.). **20**: 579 ('39). Discusses treatment of water for use in dye-works. Most important processes are neutral softening (substitution of alkali ions for calcium and magnesium ions) and breakdown of bicarbonates. For neutral softening, synthetic resins are preferable to older types of base-exchange material as they have higher base-exchange capacity, react more rapidly, have good filtering properties, may be obtained in form of granules of any desired size, and are resistant to erosion in filter. Removal of bicarbonates may be carried out by adding lime, when calcium carbonate is precipitated, or by adding acid and driving off the carbonic acid formed, or by passing water over hydrogen-exchange resins and driving off carbonic acid formed. If chlorides or sulfates are present with bicarbonates, free acids will be formed in water. To avoid this, water may be divided into two parts, of which one is passed through hydrogen-exchange filter and other through sodium-exchange filter. When two parts are mixed,

free acid in water from hydrogen-exchange filter reacts with sodium bicarbonate in water from sodium-exchange filter to form sodium salts and carbonic acid, which can be driven off. If water contains much carbonate and low concentration of neutral salts it may be passed through hydrogen-exchange and sodium-exchange filters in series. The free acids formed in first filter are, with exception of carbonic acid, converted to sodium salts in second filter. This arrangement acts as a buffer-filter for control of pH value. For complete removal of salts, water is first passed through a hydrogen-exchange filter with production of acids, and then through a hydroxyl-exchange filter which reacts with free acid to give water. As water obtained is not buffered, treatment may be followed by passage through buffer-filter. Care must be taken to protect apparatus from corrosion by acids produced in hydrogen-exchange filters.—*W. P. R.*

Some of the Factors Influencing the Selection of Mill Sites in the South. D. G. Moon. Paper Tr. Jour. **109:** TAPPI Sect. 160 ('39). Summarizes factors which must be considered when choosing site for pulp and paper mill, including water supply available. If artesian water is available, its use will probably be considered first. Water from artesian wells usually contains from 5 to over 20 g.p.g. hardness, and requires softening before being used in bleaching operations. May also be necessary to remove hydrogen sulfide and iron. Quantities of water required, and costs of obtaining it are discussed. Waters from rivers and streams not affected by salt from sea are usually soft, but require treatment to remove suspended matter. In mountainous regions soft, clear water, requiring little treatment, is often available. Underground water supplies are also available in some valleys of larger streams and in limestone formations in Ga., Ala., and Fla.—*W. P. R.*

New (Aluminum) Distilled Water System. H. V. Churchill. Chem. & Met. Eng. **46:** 4: 226 ('39). "Mallinckrodt" distilled water system for supplying water to be used in manufacture of high-purity pharmaceutical, lab., and photographic chemicals is illustrated and described. Storage-tanks, each of 2,000 gal. capac., and piping are made from "3 S" alloy (aluminum containing 1.25% manganese). Pipe fittings are made from alloy "B 214" (aluminum containing about magnesium 3.75 and silicon 1.75%). Pump and valves are of bronze. Water, after discharge from still, is aerated, as this minimizes reaction between distilled water and aluminum.—*I. M.*

ALGAE CONTROL

Algae Control. J. B. Baty. Can. Engr., Water and Sewage. **78:** 6: 20 (Jun. '40). Brief general discussion of troubles caused by algae and methods for their control. Tables are included showing amount of copper sulfate and chlorine required to destroy various organisms and amount of copper sulfate withstood by different species of fish. Most fish are able to tolerate 1 p.p.m. free chlorine for few hours and 0.3 p.p.m. indefinitely.—*R. E. Thompson.*

Chlorination of Water for Control of Mussels and Algae. R. W. Henry. Commonwealth Engr. (Australia). **26:** 72 ('38). Discusses chlorine control

of algae and molluscs in industrial water supply systems, with reference to operating experiences at several British, American, and Australian plants. Mussels may cause serious blockage in pipe lines. Growth may be prevented by continuous addition of about 0.5 p.p.m. chlorine to water during spawning season; established growths are unaffected and must be removed mechanically. Algal slimes on walls of condenser tubes greatly lower efficiency of power plants. Intermittent chlorination is usually adopted to control development of algae. Application of chlorine for control of mussels and algae is comparatively new development in Australia. Plants where chlorination has been adopted report increases in efficiency and reductions in operating costs. Chloramines formed by treating water with ammonia and chlorine are preferable to chlorine when water is acid, or when a closed circulation system is to be kept free of algae. Chloramines are less readily oxidized by organic matter in water than chlorine. Greater stability makes them particularly applicable to systems in which circulating water has comparatively high temp.—*W. P. R.*

A Four-Year Record of Ultra-Violet Energy in Daylight. MATTHEW LUCKIESH, A. H. TAYLOR AND G. P. KERR. J. Franklin Inst. **228**: 425 (Oct. '39). Results of 4-yr. study of ultra-violet energy in daylight reported, study made on outskirts of Cleveland. Ultra-violet energy measured by a cadmium-magnesium alloy phototube sensitive only to energy of wave-lengths shorter than approx. γ 3,350. Spectral sensitivity of tube such that it responds to ultra-violet wave-lengths approx. in accordance with their effectiveness in producing sunburn. On the ave., total ultra-violet energy in daylight for June is approx. 9 times as much as in Jan. and 17 times as much as in Dec. Approx. 80% of total erythemal (sunburn) ultra-violet energy for yr. is received in 6-mo. period Apr.-Sept. inclusive, only 7% in 4-mo. period Nov.-Feb.—*Martin E. Flentje.*

Sun Ray Counts Save Sulfate. R. F. GOUDEY. Eng. News-Rec. **124**: 525 (Apr. 11, '40). Los Angeles reservoirs are treated with copper sulfate for algae control when counts are high or are rapidly increasing coincident with high ultra-violet ray radiation. Conversely, when ultra-violet ray radiation is low or intensity is waning, copper sulfate is not applied even at high counts. Records indicate no direct relationship between algal population and water temp. or hr. of sunshine. Facts are shown in curves. Effect of sunshine depends on its ultra-violet content. Radiation on typical days in spring, summer, fall, and winter is also shown by means of curves. Both length of exposure per day and intensity of ultra-violet rays decrease in that order. Several days of high radiation are required to cause algal growths to increase unduly. Device used for recording ultra-violet ray radiation has been improved considerably. Since '33-'34, consumption of copper sulfate has steadily decreased from 107.5 to 60 tons per yr. and saving effected has amounted to $5,000 annually. Copper sulfate is applied in powdered form, using portable blower. This method reduces time required for treatment to ⅓ of that required for treatment by dragging sacks of crystals alongside boats.—*R. E. Thompson.*

The Operation of a Swimming Pool. WAYNE A. BECKER. Ohio Conf. Water Purification, 19th Ann. Rept. 17 ('39). Algae growths, due to spores carried in by rain, frequently develop in swimming pools 24-48 hr. after rainfall. Continuous chlorination during rainy periods, even when bathing load is light, has been found effective in preventing condition. Copper sulfate added to alkaline pool water is soon rendered ineffective by precipitation. Effectiveness may be increased by adding, prior to application, an excess of ammonia, dissolving precipitate which first appears, and forming a deep blue colored solution. A coagulant should be added to filter influent for 12-24 hr. after such treatment. In general, use of coagulants is not recommended unless use is indicated by turbidity of pool water. Presence of algae in pool water reduces chlorination efficiency, and high bacterial counts and coliform bacteria may be found in presence of 0.3-0.5 p.p.m. free chlorine. Rapid dissipation of free chlorine in pool water is a danger signal. Adequate recirculation is of utmost importance. *Discussion.* F. J. MCINTYRE: In Columbus, copper sulfate treated with ammonia as above is employed in conjunction with chloramine for algae control. 2 lb. copper sulfate per 100,000 gal. of pool water is used 2-3 times per week. When chloramine is used, min. residual chlorine content should be 0.5 p.p.m., and sometimes 1.0-1.5 p.p.m. must be maintained to ensure good bacterial results. E. T. EDWARDS: In Ironton, algae are controlled by maintaining residual chlorine content of 0.3-0.5 p.p.m. and using wall brush and vacuum cleaner once each week. High bacterial counts and coliform bacteria are found occasionally even with residual chlorine content of 0.2-0.5 p.p.m. Alkalinity is maintained at 35-55 p.p.m. F. S. TAYLOR: At Defiance, alum is applied at rate of 3-4 g.p.g. for 15 min. after backwashing filters and then dosage is reduced to very small amount. Sterility is secured when residual chlorine is in excess of 0.3 p.p.m. PHILIP J. O'CONNOR: Sodium hypochlorite and ammonium sulfate are employed at Warren, maintaining residual chlorine content of 0.5 p.p.m. Alum is applied only after backwashing filters.—*R. E. Thompson.*

A Biological Survey of the Allegheny and Chemung Watersheds. VIII. A Limnological Study of Chatauqua Lake. W. L. TRESSLER AND RUBY BERE. N. Y. State Conserv. Dept. Biol. Survey No. 12 (1938) p. 196. Lake was treated with 10 tons of $CuSO_4$ by towing 100-lb. bags of chemical through water behind a steamer. Concn. of $CuSO_4$ in soln. path, approx. 50′ wide, was about 11-35 lb./mil. gal. Counts before and after treatment showed increases in all groups of both micro- and macro-plankton except protozoa, which decreased. Other treatments, with as much as 82 lb. $CuSO_4$ per mil. gal., increased microplankton groups except green algae, which decreased considerably. Of macroplankton groups, *nauplii* increased in nos. but *Cladocera* and *Rotifera* decreased. Treatments had little, if any, effect on fish life. Surface sample of water from center of southern half of lake, before treatment with $CuSO_4$ on June 24, contained 0.068 p.p.m. of Cu. On July 19, a mixt. of surface samples from northern and southern parts of the lake contained 0.114 p.p.m. and on Sept. 1, a bottom sample taken off Bemus Point Fish Hatchery at 5 m. depth contained 0.108 p.p.m.—*C. A.*

ADMINISTRATION

The Changing Incidence of Public Utility Taxation. Jesse V. Burkhead. J. Land & Pub. Util. Econ. **15:** 383 (Nov. '39). Problems in incidence of public utility taxes have been generally considered easier of solution than problems in incidence of other business taxes. Rates charged by public utilities fixed by regulatory commissions allowing a fair return on all operating expenses including taxes. During rapid economic changes, tax burden may fall on ownership due to inherent lag in regulatory process. If utility is charging full monopoly price increased taxes will fall on ownership. After acceptance of "regulated incidence" by courts and commissions, it was generally followed until beginning of depression, since increased taxes burden on ownership, increase lowered rate of return on investment. After Smyth-Ames' doctrine of fair return on fair valuation became part of commission procedure, imposition of taxes thereafter would not be "fair." Mr. Justice Brandeis speaking for Supreme Court stated all taxes including income taxes are operating expense. Increased taxes due to search by state legislatures and Congress for stable revenue, taxes on consumption. In depression, consumers adverse to increased rates, additional taxes on utilities, rate schedules constant, increased taxes reduced return on utility investment. Interest rates declined in last decade. Commission relied less on valuation and more on bargaining and competition in rate regulation. Equity of results obtained through utility taxation depends not only on burden in each particular case but on customers' ability to pay. Inadequate regulation plus special taxes more equitable than inadequate regulation and no special taxes, less equitable than adequate regulation and no special taxes. New York Supreme Court in reversing New York Public Service Commission decision ruled: "If 6 per cent is a reasonable rate of return, any order of reduction that will deprive petitioner of such reasonable return is confiscatory." Public utility rate structures more inelastic than other price and cost structures. Taxes imposed at time when there is no possibility of lower rates, later may be used by utilities to defer rate reductions benefiting entire community. Four conditions between utility regulation and taxation: (1) good regulation and no special taxes; (2) poor regulation and no special taxes—undesirable, consumers pay excessive rates, no benefit to community; (3) poor regulation and reasonable and temporary taxes; (4) poor regulation and unreasonable and permanent taxes. Danger is imminent of devising special permanent taxes. Used together, taxation and regulation can equitably solve conflicting interests in public utility relationships.—*Samuel A. Evans.*

The Valuation of Water Undertakings for Rating Purposes. W. Read Ward. Wtr. and Wtr. Eng. (Br.) **42:** 146, 185, 217, 252 (Apr., May, June, July '40). Surveyor (Br.) **97:** 213 (Mar. 1, '40). Taxes exercise such a profound influence on water works administration that activities and enterprise are sometimes seriously handicapped due to depletion of potential financial resources. Taxing of water works is governed principally by 2 acts: The Rating and Valuation Act of 1925, and The Valuation (Metropolis) Act of

1869. So many difficulties have arisen in application of acts that recourse to Courts for interpretations has been necessitated from time to time. Inquiry might result in development of alternative means of achieving desired end, unhampered by "the mortmain of past authority." 3 principal methods have been applied in evaluating water works: (1) Comparative Method, which consists of assessing similar properties at like valuation; (2) Structural Method, which assigns assessment based on present effective capital value of land and buildings; and (3) Accountancy Method, which takes into account receipts and expenses of enterprise. Difficulties to be overcome are many. A larger capital outlay is required under gravitational scheme of layout than pumping scheme, and a larger income has to be provided. The income comes under contribution, when undertaking is valued for taxing purposes, to initial disadvantage of gravitational scheme. Working expenses, however, are usually much lower than those attached to a pumping scheme. Economies effected in the hope of paying way without loss have unfortunate effect of automatically increasing taxable value. Rate of interest on money borrowed is dependent on market conditions, credit of the undertaking, and security offered. Amount of capital required to be raised will be practically identical with construction cost of works. Interest would be payable on whole of money borrowed and would, to extent of surplus capital involved, be in excess of that payable in respect of expenditure or actual tangible assets. Gross receipts would thus be inflated and additional income would come under contribution for taxing. Present system penalizes undertaking that is hardest hit and favors that with the least obligations. Distinction has to be made between sinking fund, or replacement charges as a statutable deduction in computing taxable value of an undertaking, and sinking fund charges representing provision for repayment of capital borrowed. Period over which these capital replacements have to be discharged does not correspond with "life" allowed under head of statutables—replacement of works. Necessary to distinguish between effective replacement capital value and constructional cost. Former consists of estimate of cost of replacing works, having regard to present day prices. Construction costs are those that appear in accounts of original undertaking. Restrictions imposed by statute are, at times, placed upon profit earning capacity of undertakings, and law requires that due regard shall be given to such restrictions in assessing taxable value. Neither rent paid by a tenant, nor actual cost of a hereditament, is not which necessarily satisfies requirements of Rating and Valuation Act as "rent" for rating purposes. No attempt has been made to discredit conclusions arrived at under profits method. Remarks are presented with view to exemplifying inherent difficulties in profits method and to show what any new proposals must avoid. Up to present, only constructive alternative to accountancy method that has been applied is that known as contractor's test, i.e., by taking replacement value of the works as a basis, and "applying thereto an appropriate rate per cent of interest on capital represented which would remunerate the contractor for providing the properties." Further possible expedient might be adopted with view to assuring more or less uniform treatment by adjusting allowances made under accountancy method by adjusting: interest charges, sinking fund for depreciation, tenant's share, working expenses, taxes, excess

works, and gross receipts. Judgments in Marleybone and Peterborough cases suggest that, in those cases of municipal undertakings which have power of precepting for making good deficiencies, a possible means of avoiding excessive tax payments might legally be found by so reducing charges for water that insufficient receipts would be available to meet all expenditures. Local authorities might thus be called upon to contribute to income of water company instead of receiving taxes from it. Accountancy method can be expressed algebraically as $365PQ = R + A$, and under the proposed new method $R = QX$ in which P = price per 1,000 gal.; Q = ave. daily quantity supplied in 1,000 gal.; R = rent or ratable value; A = authorized allowance made under accountancy method; and X = factor (to be ascertained) to be applied under new method for converting quantity into rent or taxable value. Application of proposed new method does no violence to principles laid down in tax law. Distribution of capital replacement value could be made on somewhat similar lines to those at present applied, but subject throughout to governing factor of user. Likewise, method of apportionment should be effected on basis of user but with respect to certain classes of hereditaments, it might be difficult to give effect to such a method, since necessary data might not be available. Such hereditaments include impounding reservoirs, service reservoirs, pumping stations, trunk mains, and service mains. Among advantages of adoption of new method may be included: sum arrived at would bear proper relation to utility undertaking contributes to community; each hereditament would be valued for taxing in proportion to use it contributed to whole undertaking; no disparity in taxable value would occur between gravitational and pumping schemes; sinking fund contributions would not enter into consideration; all undertakings would be assessed on a uniform and definite basis; it would avoid possibility of reducing taxable value by awarding too much to tenant; it would avoid all of the difficulties inherent in the accountancy method; and it would obviate necessity for *ad interim* proposals being submitted for new works.—*H. E. Babbitt.*

California Railroad Commission. ***Re: Independence Acres Water Works.*** Commission Order. Pub. Util. Fort. (Dec. 7, '39), P.U.R. **30**: 293. Due to inadequate quantity of water, water company appeared before commission asking that number of consumers be limited to those who had made written application before date of hearing and those having actually under construction residences in the tract. Wells had been drilled and best engineering advice followed in an endeavor to procure water. Commission ordered that water service be restricted to lots heretofore served, lots for which applications for water service have been made and to such lots upon which buildings are actually under construction on date of order. Water company file with commission names of all consumers and applications for 2 years previous to order.—*Samuel A. Evans.*